STRUCTURAL ENGINEERS' HANDBOOK

The quality of the materials used in the manufacture of this book is governed by continued postwar shortages.

STRUCTURAL ENGINEERS' HANDBOOK

DATA FOR THE DESIGN AND CONSTRUCTION OF STEEL BRIDGES AND BUILDINGS

BY

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PREFACE

The aim in writing this book has been to give data, details and tables for the design and construction of steel bridges and buildings. The book is written for the structural engineer and for the student or engineer who has had a thorough course in applied mechanics and the calculation of stresses in structures. To this end data and tables that will be of service to the designing and constructing engineer have been given, rather than predigested data and designs that might be used by the untrained. The book is intended as a working manual for the engineer, draftsman and student and covers data, details and tables for the design of the structures ordinarily met with. Swing and movable bridges, cantilever and suspension bridges require special treatment and have not been considered. As the book is intended to supplement the present books on stresses the calculation of stresses in bridges and buildings has been only briefly considered. The calculation of stresses in retaining walls, bins, stand-pipes, and other structures not ordinarily covered in text-books on stresses have been given in compact form. Great care has been used to give examples of structures that represent standard practice. With a few exceptions the drawings of details of structures have been especially prepared for this book from actual working plans. The book is a source book and is not a treatise, and is intended to furnish data and details that are available only to a few engineers; and standard specifications for materialsand workmanship that are available only in transactions of societies and in special treatises.

The tables giving properties of columns, top chords, plate girders and struts have been calculated especially for this book, and are original in material and arrangement. In calculating the tables only those sections which comply with standard specifications have been given. The tables have been calculated by the use of calculating machines and have been checked with great care. The values will be found to be correct to one unit in the last place given. Properties of Carnegie and Bethlehem sections are given in a compact form for easy reference. The tangents of the angle of the axis giving the least radius of gyration, given in the tables giving properties of Carnegie angles, were taken from Cambria Steel. With the exception of a few special I beams and channels the tables may be used for Cambria, Pencoyd and Jones & Laughlin angles, I beams and channels. The American Bridge Company standards for eye-bars, loop-bars, clevises, pins, and other structural details are given. Tables of logarithms, function of angles and tables that are easily available have not been included.

The size of the book and the size of the type page were selected for the reasons that they give a book of standard size with a type page large enough so that each table can come squarely on one page, and large enough so that complete plans of structures can be given. A large clear type was selected for both the text and for the tables. The paper has been selected with the idea of clearness of the printed page.

This book is a result of many years' work, during which time the author has written four books on structural engineering. In writing this book the author has drawn on his other books, although much of the material given on steel mill buildings and highway bridges is new, and the Structural Engineers' Handbook supplements the author's other books.

Data and details have been obtained from many sources, to which credit has been given in the body of the book. The author is under special obligation to many engineers, to which special acknowledgment cannot be made on account of lack of space.

In writing this book the author has been assisted by several of his former students. Credit is due to Mr. I. C. Crawford, Instructor in Civil Engineering, for assistance in calculating tables and reading proof; to Mr. C. S. Sperry, Instructor in Engineering Mathematics, for assistance in calculating tables; to Professor H. C. Ford, of Iowa State College, and Mr. T. A. Blair, Instructor in Civil Engineering, for assistance in preparing the drawings; and especially to Mr. W. C. Huntington, Assistant Professor of Civil Engineering, for assistance in arranging and calculating tables, reading proof and assistance in other ways.

The author will appreciate notices of errors and suggestions for the improvement of future editions.

M. S. K

BOULDER, COLORADO.
August 23, 1914.

PREFACE TO SECOND EDITION

In this edition details of steel windows and doors, data on cement and gypsum tile roofs, solutions for bending moments in mill building columns and stresses in stiff frames have been added to Chapter I, and Chapter III, Steel Highway Bridges, has been rewritten and enlarged. All known errors have been corrected. Duties required of the author as Assistant Director in Charge of Construction of the U. S. Government Explosives Plant, Nitro, West Virginia, have made it impossible to complete a more thorough revision that was planned.

M. S. K.

U. S. GOVERNMENT EXPLOSIVES PLANT "C,"
NITRO, WEST VIRGINIA,
May 12, 1918.

PREFACE TO THIRD EDITION

In this edition the book has been revised and partially rewritten, and more than 130 pages of new material has been added. The most important additions are Chapter XIA, " Design of Self-supporting Steel Stacks," the American Bridge Company's standards for " Constant Dimension Steel Columns " and " Steel Column Footings," and the American Institute of Steel Construction, " Specifications for Structural Steel for Buildings." Other important additions and revisions are: Revised specifications for steel frame buildings and steel highway bridges; A.R.E.A., 1920, " General Specifications for Steel Railway Bridges "; revised A.S.T.M. specifications for engineering materials; A.R.E.A., 1923, " Specifications for the Erection of Steel Railway Bridges "; A.R.E.A. " Specifications for Concrete, Plain and Reinforced "; additional data on steel mill buildings, steel office buildings, steel and timber highway bridges, steel railway bridges, and retaining walls; stresses in stiff frames and in eccentric riveted connections.

The tables in Part II have been revised to comply with the new standards adopted by the Association of American Steel Manufacturers, and data and details of the latest standard Carnegie and Bethlehem sections have been provided. New data for electric traveling cranes, and tables for the calculation of the stresses in eccentric riveted connections are given. By permission of the American Bridge Company the details and properties of " Constant Dimension Steel Columns " and " Steel Column Footings," covering 29 pages, are given in Part II.

The Author wishes to acknowledge the appreciation with which former editions have been received by teachers, students and engineers.

M. S. K.

URBANA, ILLINOIS,
July 1, 1924.

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STRUCTURAL ENGINEERS' HANDBOOK

Introduction.—The book is divided into two parts which are self contained. Part I includes a discussion of the design of structures and gives data and details for the design of steel bridges and buildings. Part II contains tables for structural design and includes tables giving the properties of rolled sections, properties of built-up sections for chords, columns, struts, plate girders, etc., and data for standard structural details.

PART I.

Data and Details for the Design and Construction of Steel Bridges and Buildings.

Introduction.—The discussion in Part I has been limited to steel bridges and buildings and other simple steel structures; no reference being made to swing and movable bridges, cantilever and suspension bridges. The design of a bridge includes the design of the substructure as well as the superstructure, so that the design of retaining walls and bridge abutments has been briefly discussed. Timber trestles and bridges are required for temporary structures and for the erection of steel structures, and a brief discussion of timber trestles and bridges is therefore properly included.

The design of a structure requires not only a knowledge of the properties of materials and the ability to calculate the stresses, but also a knowledge of local conditions and requirements, of economic design, of details of construction, of methods of erection, methods of fabrication and their effect on cost, and of many other matters which limit the design. The most economical structure for any given conditions is the one which will give the greatest service for the least money, quality of service and the life of the structure being given proper consideration. Financial limitations often limit the design and the problem then is to design a structure that will give satisfactory service with the money available.

To design a satisfactory structure when limited by financial considerations is a problem that requires the exercise of the highest possible skill on the part of the engineer. He must be able to select an economical type of structure; he must make an accurate estimate of the loads to be carried by the structure; he must be able to calculate the stresses with accuracy; he must make the detailed design with due reference to ease of obtaining the material, the cost of shop work, and the cost of erection.

The shop cost of steel structures varies with the type of structure, the size and weight of the members and upon the make-up of the members and the details. By using fewer and larger members, by using rolled beams and columns in the place of built-up plate girders and columns, and by using tie plates in the place of lacing, the shop cost per pound of a railroad bridge may be materially reduced. If the simplification of the design is carried too far the reduction in shop cost will result in a material increase in the weight of the bridge, and in an increase in the cost of the bridge, with a decrease in efficiency. The details of the design of a structure should be worked out with reference to ease and economy of erection as well as ease and low cost of fabrication. While the standardizing of connections so that multiple punches may be used may result in a considerable

saving in shop cost, it often results in a material increase in the weight of the details of the structure, and in the number of field rivets, so that the efficiency of the structure is not increased, and the final cost of the structure is not reduced. The author has in mind a case where to change the details of a plate girder so that multiple punches might be used required the addition of details equal to 5 per cent of the weight of the span and the addition of 25 per cent to the number of field rivets, with no increase in efficiency.

The best results are obtained when the structural engineer prepares carefully worked out detail drawings (not shop drawings) in which the efficiency of the structure, ease of fabrication and ease of erection are given due consideration. The shop drawings may then be prepared by the bridge company to take the greatest possible advantage of improved shop methods without decreasing the efficiency of the structure, or increasing the total weight, or increasing the cost of erection.

Part I is divided into seventeen chapters, of which the first eleven chapters cover different types of structures, and the last six chapters cover subjects which apply to all types of steel construction. While the aim has been to present the largest possible amount of information in the limited space, each subject presented is discussed briefly in a logical order.

While the author has drawn on his other books in the various chapters, the reader will find much new material on the subjects covered in the other books, especially in Chapter I, Steel Roof Trusses and Mill Buildings, and Chapter III, Steel Highway Bridges, so that this book supplements the author's other books on structures. Each chapter is self-contained, the illustrations and tables being numbered independently of the other chapters. As far as possible the different subjects are discussed fully in each chapter, thus reducing cross-references. The most of the cross-referencing is made through the index, which together with the table of contents will be found invaluable to the reader.

CHAPTER I.

Steel Roof Trusses and Mill Buildings.

Definitions.—The following definitions will assist the reader in a study of roof trusses and steel frame buildings.

Truss.—A truss is a framed structure in which the members are so arranged and fastened at their ends that external loads applied at the joints of the truss will cause only direct stresses in the members. In its simplest form a truss is a triangle or a combination of triangles. In this chapter it will be assumed (1) that the structure is not constrained by the reactions, (2) that the axes of the members meet in a common point at the joints, and (3) that the joints have frictionless hinges.

Transverse Bent.—A transverse bent consists of a truss supported at the ends on columns and braced against longitudinal movement by knee braces attached to the lower chord of the truss and to the columns.

Purlin.—A beam that rests on the top chords of roof trusses and supports the sheathing that carries the roof covering, or supports the roof covering directly, or supports rafters.

Rafter.—A beam that rests on the purlins and supports the sheathing, or may support sub-purlins. Rafters are not commonly used in mill buildings.

Sub-purlin.—A secondary system of purlins that rest on the rafters and are spaced so as to support the tile or slate covering directly without the use of sheathing.

Sheathing.—A covering of boards or reinforced concrete that is carried on the purlins or rafters to furnish a support for the roof covering.

Girt.—A beam that is fastened to the columns to support the side covering either directly or to support the side sheathing.

Monitor Ventilator.—A framework at the top of the roof that carries fixed or movable louvres, or sash in the clerestory.

Clerestory.—The clear opening in the side framework of a monitor ventilator of a building, also the clear opening on the side of a building.

Louvres.—Slats made of metal or wood which are placed in the clerestory of a monitor ventilator to keep out the storm. Louvres may be fixed or movable. The opening of a monitor ventilator is also called a louvre.

Panel.—The distance between two joints in a roof truss or the distance between purlins.

Bay.—The distance between two trusses or transverse bents.

Pitch.—The pitch of a truss is the center height of the truss divided by the span where the truss is symmetrical about the center line.

Other terms are defined when they are first used.

Data for the Design of Roof Trusses and Steel Frame Buildings.

Weight of Roof Trusses.—The weight of roof trusses varies with the spa[illegible] distance between trusses, the load carried or capacity of the truss, and the pitch.

The empirical formula

$$W = \frac{P}{45} A \cdot L \left(1 + \frac{L}{5\sqrt{A}} \right) \qquad (1)$$

where

W = weight of steel roof truss in pounds;
P = capacity of truss in pounds per square foot of horizontal projection of roof (30 to 80 lb.);
A = distance center to center of trusses in feet (8 to 30 ft.);
L = span of truss in feet;
was deduced by the author from the computed and shipping weights of mill building trusses of the Fink type.

Weight of Purlins, Girts, Bracing, and Columns.—Steel purlins will weigh from 1½ to 4 lb. per sq. ft. of area covered, depending upon the spacing and the capacity of the trusses and the snow load. Girts and window framing will weigh from 1¼ to 3 lb. per sq. ft. of net surface. Bracing is quite a variable quantity. The bracing in the planes of the upper and lower chords will vary from ½ to 1 lb. per sq. ft. of area. The side and end bracing, eave struts and columns will weigh about the same per sq. ft. of surface as the trusses.

Weight of Roof Covering.—The weight of corrugated iron or steel covering varies from 1½ to 3 lb. per sq. ft. of area. The weight of corrugated steel is given in Table I. The approximate weight per square foot of various roof coverings is given in the following table:

Corrugated steel, without sheathing	1 to 3 lb.
Felt and asphalt, without sheathing	2 "
Tar and Gravel Roofing, without sheathing	8 to 10 "
Slate, 3/16 in. to ¼ in., without sheathing	7 to 9 "
Tin, without sheathing	1 to 1½ "
Skylight glass, 3/16 in. to ½ in., including frames	4 to 10 "
White pine sheathing 1 in. thick	3 "
Yellow pine sheathing 1 in. thick	4 "
Tiles, flat	15 to 20 "
Tiles, corrugated	8 to 10 "
Tiles, on concrete slabs	30 to 35 "
Plastered ceiling	10 "

The actual weight of roof coverings should be calculated if possible.

Snow Loads.—The annual snowfall in different localities is a function of the humidity and the latitude and is quite a variable quantity. The amount of snow on the ground at one time is still more variable. The snow loads given in Fig. 1 were proposed by the author in "The Design of Steel Mill Buildings" in 1903 and have been generally adopted.

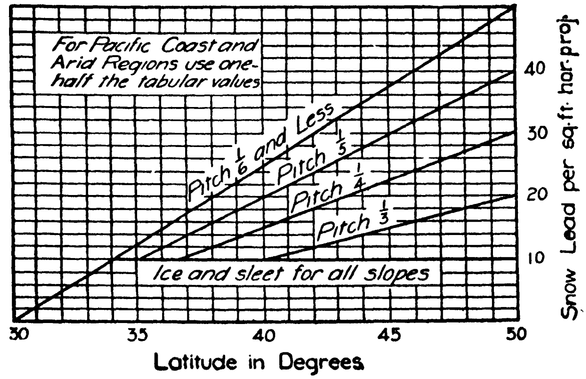


FIG. 1. SNOW LOAD ON ROOFS FOR DIFFERENT LATITUDES, IN POUNDS PER SQUARE FOOT.

One of the heaviest falls of snow on record occurred at Boulder and Denver, Colorado on Dec. 5 and 6, 1913, when 36 inches of snow weighing 9 lb. per cu. ft. fell during two days. Many

flat roofs were loaded with a snow load of more than 30 lb. per sq. ft. and roofs with a pitch of one-half carried the full snow load of 27 lb. per sq. ft. of horizontal projection.

A high wind may follow a heavy sleet and in designing the trusses the author would recommend the use of a minimum snow and ice load as given in Fig. 1 for all slopes of roofs. The maximum stresses due to the sum of this snow load, the dead and wind loads; the dead and wind loads; or of the maximum snow load and the dead load being used in designing the members.

Wind Loads.—The wind pressure, P, in pounds per square foot on a flat surface normal to the direction of the wind for any given velocity, V, in miles per hour is given quite accurately by the formula

$$P = 0.004\ V^2 \tag{2}$$

The pressure on other than flat surfaces may be taken in per cents of that given by formula (2) as follows: 80 per cent on a rectangular building; 67 per cent on the convex side of cylinders; 115 to 130 per cent on the concave side of cylinders, channels and flat cups; and 130 to 170 per cent on the concave sides of spheres and deep cups.

Recent German specifications for design of tall chimneys specify wind loads per square foot as follows: 26 lb. on rectangular chimneys; 67 per cent of 26 lb. on circular chimneys; and 71 per cent of 26 lb. on octagonal chimneys.

The official specifications for the design of steel framework in Prussia have recently been amplified in the matter of wind pressures. For the wind-bracing, as a whole, the wind pressure on the whole building is to be taken as 17 lb. per sq. ft. For proportioning individual frame members, girts, studs, trusses, etc., a higher value of wind pressure must be assumed, viz., 28 to 34 lb. per sq. ft.

It would seem that 30 lb. per square foot on the side and the normal component of a horizontal pressure of 30 lb. on the roof would be sufficient for all except exposed locations. If the building is somewhat protected a horizontal pressure of 20 lb. per square foot on the sides is certainly ample for heights less than, say 30 feet.

Wind Pressure on Inclined Surfaces.—The wind is usually taken as acting horizontally and the normal component on inclined surfaces is calculated.

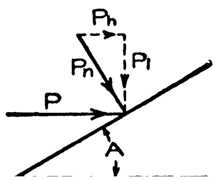


FIG. 2.

The normal component of the wind pressure on inclined surfaces has usually been computed by Hutton's empirical formula

$$P_n = P \cdot \sin A^{1.842 \cos A - 1} \tag{3}$$

where P_n equals the normal component of the wind pressure, P equals the pressure per square foot on a vertical surface, and A equals the angle of inclination of the surface with the horizontal, Fig. (2).

The formula due to Duchemin

$$P_n = P \frac{2 \sin A}{1 + \sin^2 A} \tag{4}$$

where P_n, P and A are the same as in (3), gives results considerably larger for ordinary roofs than Hutton's formula, and is coming into quite general use.

The formula

$$P_n = P \cdot A/45 \tag{5}$$

where P_n and P are the same as in (3) and (4), and A is the angle of inclination of the surface in degrees (A being equal to or less than 45°), gives results which agree very closely with Hutton's formula, and is much more simple.

Hutton's formula (3) is based on experiments which were very crude and probably erroneous. Duchemin's formula (4) is based on very careful experiments and is now considered the most reliable formula in use. The Straight Line formula (5) agrees with experiments quite closely and is preferred by many engineers on account of its simplicity.

The values of P_n as determined by Hutton's, Duchemin's and the Straight Line formulas are given in Fig. 3, for P equals 20, 30 and 40 lb.

It is interesting to note that Duchemin's formula with P equals 30 pounds gives practically the same values for roofs of ordinary inclination as is given by Hutton's and the Straight Line formulas with P equals 40 pounds.

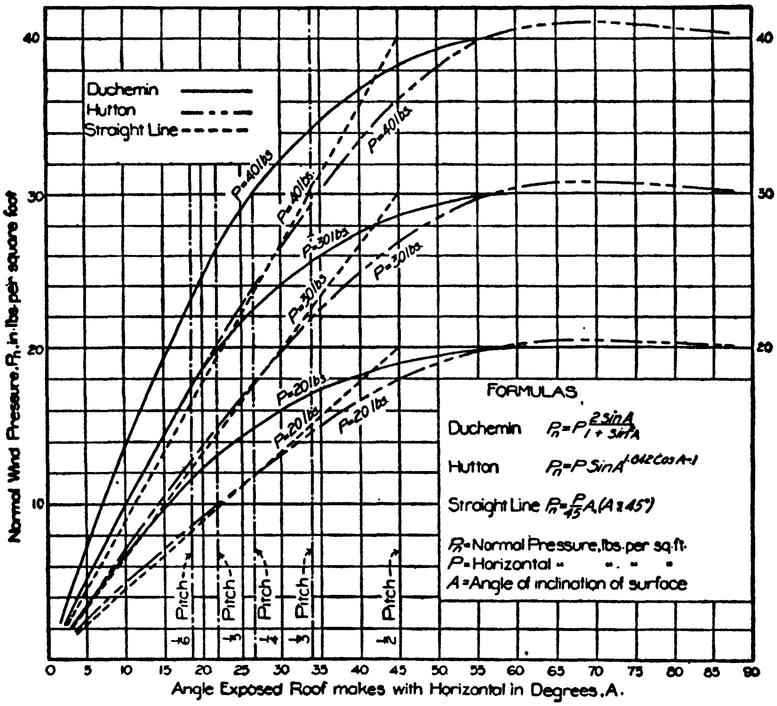


FIG. 3. NORMAL WIND LOAD ON ROOF ACCORDING TO DIFFERENT FORMULAS.

Duchemin has also deduced the formula

$$P_h = P\frac{2\sin^2 A}{1+\sin^2 A} \tag{6}$$

where P_h in (6) equals the pressure parallel to the direction of the wind, Fig. 2; and

$$P_l = P\frac{2\sin A\cdot\cos A}{1+\sin^2 A} \tag{7}$$

where P_l in (7) equals the pressure at right angles to the direction of the wind, Fig. 2. P_l may be an uplifting, a depressing or a side pressure. With an open shed in exposed positions the uplifting effect of the wind often requires attention. In that case the wind should be taken normal to the inner surface of the building on the leeward side, and the uplifting force determined

by using formula (7). If the gables are closed a deep cup is formed, and the normal pressure should be increased 30 to 70 per cent.

That the uplifting force of the wind is often considerable in exposed localities is made evident by the fact that highway bridges are occasionally wrecked by the wind.

The wind pressure is not a steady pressure, but varies in intensity, thus producing excessive vibrations which cause the structure to rock if the bracing is not rigid. The bracing in mill buildings should be designed for initial tension, so that the building will be rigid. Rigidity is of more importance than strength in mill buildings.

Miscellaneous Loads.—Data on the weights of materials are given in Chapter II. The weights and other data for hand cranes are given in Table 133 and of electric cranes are given in Table 130, Part II.

Minimum Loads.—For minimum loads to be calculated on roofs see § 29, "Specifications for Steel Frame Buildings" in the last part of this chapter.

STRESSES IN ROOF TRUSSES AND MILL BUILDINGS.—For the calculation of the stresses in roof trusses and in the framework of steel frame mill buildings, see the author's "The Design of Steel Mill Buildings."

Design of Steel Mill Buildings.

General Principles of Design.—The general dimensions and the outline of a mill building will be governed by local conditions and requirements. The questions of light, heat, ventilation, foundations for machinery, handling of materials, future extensions, first cost and cost of maintenance should receive proper attention in designing the different classes of structures. One or two of the above items often determines the type and general design of the structure. Where real estate is high, the first cost, including the cost of both land and structure, causes the adoption in many cases of a multiple story building, while on the other hand where the site is not too expensive the single story shop or mill is usually preferred. In coal tipples and shaft houses the handling of materials is the prime object; in railway shops and factories turning out heavy machinery or a similar product, foundations for the machinery required, and convenience in handling materials are most important; while in many other classes of structures such as weaving sheds, textile mills, and factories which turn out a less bulky product with light machinery, and which employ a large number of men, the principal items to be considered in designing are light, heat, ventilation and ease of superintendence.

Shops and factories are preferably located where transportation facilities are good, land is cheap and labor plentiful. Too much care cannot be used in the design of shops and factories for the reason that defects in design that cause inconvenience in handling materials and workmen, increased cost of operation and maintenance are permanent and cannot be removed.

The best modern practice inclines toward single floor shops with as few dividing walls and partitions as possible. The advantages of this type over multiple story buildings are (1) the light is better, (2) ventilation is better, (3) buildings are more easily heated, (4) foundations for machinery are cheaper, (5) machinery being set directly on the ground causes no vibrations in the building, (6) floors are cheaper, (7) workmen are more directly under the eye of the superintendent, (8) materials are more easily and cheaply handled, (9) buildings admit of indefinite extension in any direction, (10) the cost of construction is less, and (11) there is less danger from damage due to fire.

The walls of shops and factories are made (1) of brick, stone, or concrete; (2) of brick, hollow tile or concrete curtain walls between steel columns; (3) of expanded metal and plaster curtain walls and glass; (4) of concrete slabs fastened to the steel frame; and (5) of corrugated steel fastened to the steel frame.

The roof is commonly supported by steel trusses and framework, and the roofing may be slate, tile, tar and gravel or other composition, tin or sheet steel, laid on board sheathing or on concrete slabs, tile or slate supported directly on the purlins, or corrugated steel supported on board sheathing or directly on the purlins. Where the slope of the roof is flat a first grade tar

and gravel roof, or some one of the patent composition roofs is used in preference to tin, and on a steep slope slate is commonly used in preference to tin or tile. Corrugated steel roofing is much used on boiler houses, smelters, forge shops, coal tipples, and similar structures.

Floors in boiler houses, forge shops and in similar structures are generally made of cinders; in round houses brick floors on a gravel or concrete foundation are quite common; while in buildings where men have to work at machines the favorite floor is a wooden floor on a foundation of cinders, gravel, or tar concrete. Where concrete is used for the foundation of a wooden floor it should be either a tar or an asphalt concrete, or a layer of tar should be put on top of the cement concrete to prevent decay. Concrete or cement floors are used in many cases with good results, but they are not satisfactory where men have to stand at benches or machines. Wooden racks on cement floors remove the above objection somewhat. Where rough work is done, the upper or wearing surface of wooden floors is often made of yellow pine or oak plank, while in the better classes of structures, the top layer is commonly made of maple. For upper floors some one of the common types of fireproof floors, or as is more common a heavy plank floor supported on beams may be used.

Care should be used to obtain an ample amount of light in buildings in which men are to work. It is now the common practice to make as much of the roof and side walls of a transparent or translucent material as practicable; in many cases fifty per cent of the roof surface is made of glass, while skylights equal to twenty-five to thirty per cent of the roof surface are very common. Direct sunlight causes a glare, and is also objectionable in the summer on account of the heat. Where windows and skylights are directly exposed to the sunlight they may best be curtained with white muslin cloth which admits much of the light and shades perfectly. The "saw tooth" type of roof with the shorter and glazed tooth facing the north, gives the best light and is now coming into quite general use.

Plane glass, wire glass, factory ribbed glass, and translucent fabric are used for glazing windows and skylights. Factory ribbed glass should be placed with the ribs vertical for the reason that with the ribs horizontal, the glass emits a glare which is very trying on the eyes of the workmen. Wire netting should always be stretched under skylights to prevent the broken glass from falling down, where wire glass is not used.

Heating in large buildings is generally done by the hot blast system in which fans draw the air across heated coils, which are heated by exhaust steam, and the heated air is conveyed by ducts suspended from the roof or placed under the ground. In smaller buildings, direct radiation from steam or hot water pipes is commonly used.

The proper unit stresses, minimum size of sections and thickness of metal will depend upon whether the building is to be permanent or temporary, and upon whether or not the metal is liable to be subjected to the action of corrosive gases. For permanent buildings the author would recommend 16,000 lb. per square inch for allowable tensile, and $16{,}000 - 70\frac{l}{r}$ lb. per square inch for allowable compressive stress for direct dead, snow and wind stresses in trusses and columns; l being the center to center length and r the radius of gyration of the member, both in inches. For wind bracing and flexural stresses in columns due to wind, add 25 per cent to the allowable stresses for dead, snow and wind loads. For temporary structures the above allowable stresses may be increased 20 to 25 per cent.

The minimum size of angles should be $2'' \times 2'' \times \frac{1}{4}''$, and the minimum thickness of plates $\frac{1}{4}$ in., for both permanent and temporary structures. Where the metal will be subjected to corrosive gases as in smelters and train sheds, the allowable stresses should be decreased 20 to 25 per cent, and the minimum thickness of metal increased 25 per cent, unless the metal is fully protected by an acid-proof coating (at present the best paints do little more in any case than delay and retard the corrosion).

The minimum thickness of corrugated steel should be No. 20 gage for the roof and No. 22 for the sides; where there is certain to be no corrosion Nos. 22 and 24 may be used for the roof and sides respectively.

Steel Frame Mill Buildings.—The framework of a steel frame mill building consists of a series of transverse bents, which carry the purlins on the tops of the trusses, and girts on the sides of the columns to carry the covering, Fig, 4. The framework is braced by diagonal bracing in the planes of the roof and the sides of the building, and in the plane of the lower chords. A transverse bent consists of a roof truss supported at the ends on columns and is braced against endwise movement by means of knee braces. The framing plan for a steel frame mill building is shown in Fig. 4. Steel mill buildings are also made with end trusses in place of the end framing shown in Fig. 4.

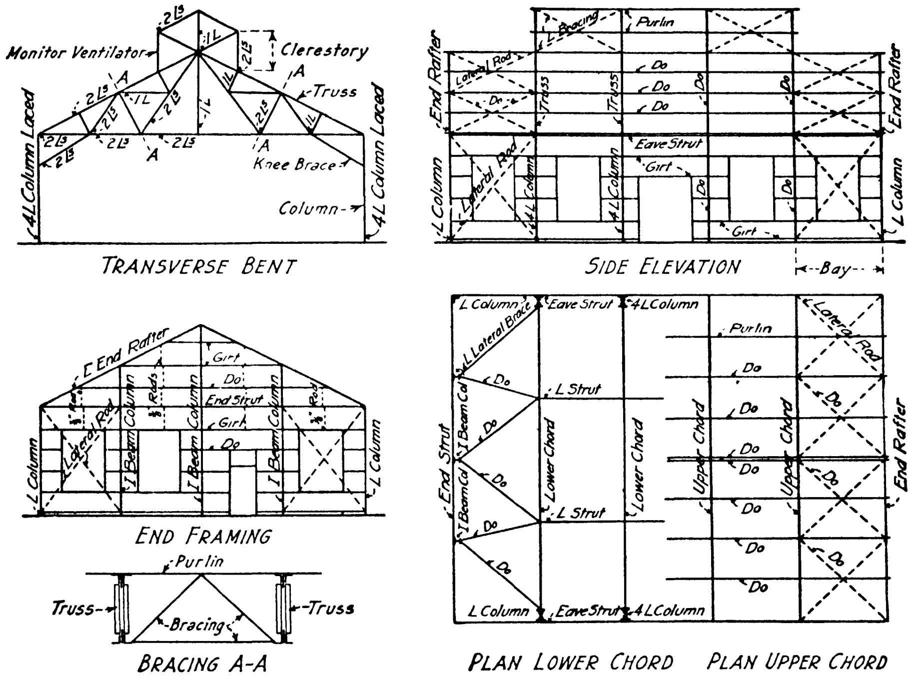


FIG. 4. FRAMEWORK FOR A STEEL MILL BUILDING.

Types of Roof Trusses.—Several types of roof trusses are shown in Fig. 5. These trusses have been subdivided so that the purlins will come at the panel points, and will not have a spacing greater than 4 ft. 9 in., the greatest spacing allowed for corrugated steel roofing when laid without sheathing. The Fink trusses shown in (*a*) to (*g*) are commonly used in steel frame buildings and are very economical. The other types of trusses need no explanation.

Different methods of lighting and ventilating buildings through the roof are shown in Fig. 6.

Saw Tooth Roofs.—The common type of saw tooth roof is shown in (*m*) Fig. 6. The glazed leg faces the north and permits only indirect light to enter the building, thus doing away with the glare and varying intensity of light in buildings where direct sunlight enters. In cold climates the snow drifts the gutters nearly full and causes loss of light and also leakage from the overflowing gutters. The modified saw tooth roof shown in (*n*) was designed by the author, to obviate the defects in the common type of saw tooth roof. The modified saw tooth roof permits the use of a greater span and more economical pitch than the common form shown in (*m*).

Transverse Bents.—A number of the common forms of transverse bents are shown in Fig. 7. Transverse bents (*a*), (*b*), (*d*), and (*h*) are used for boiler houses, shops, etc., while (*c*), (*e*), (*f*)

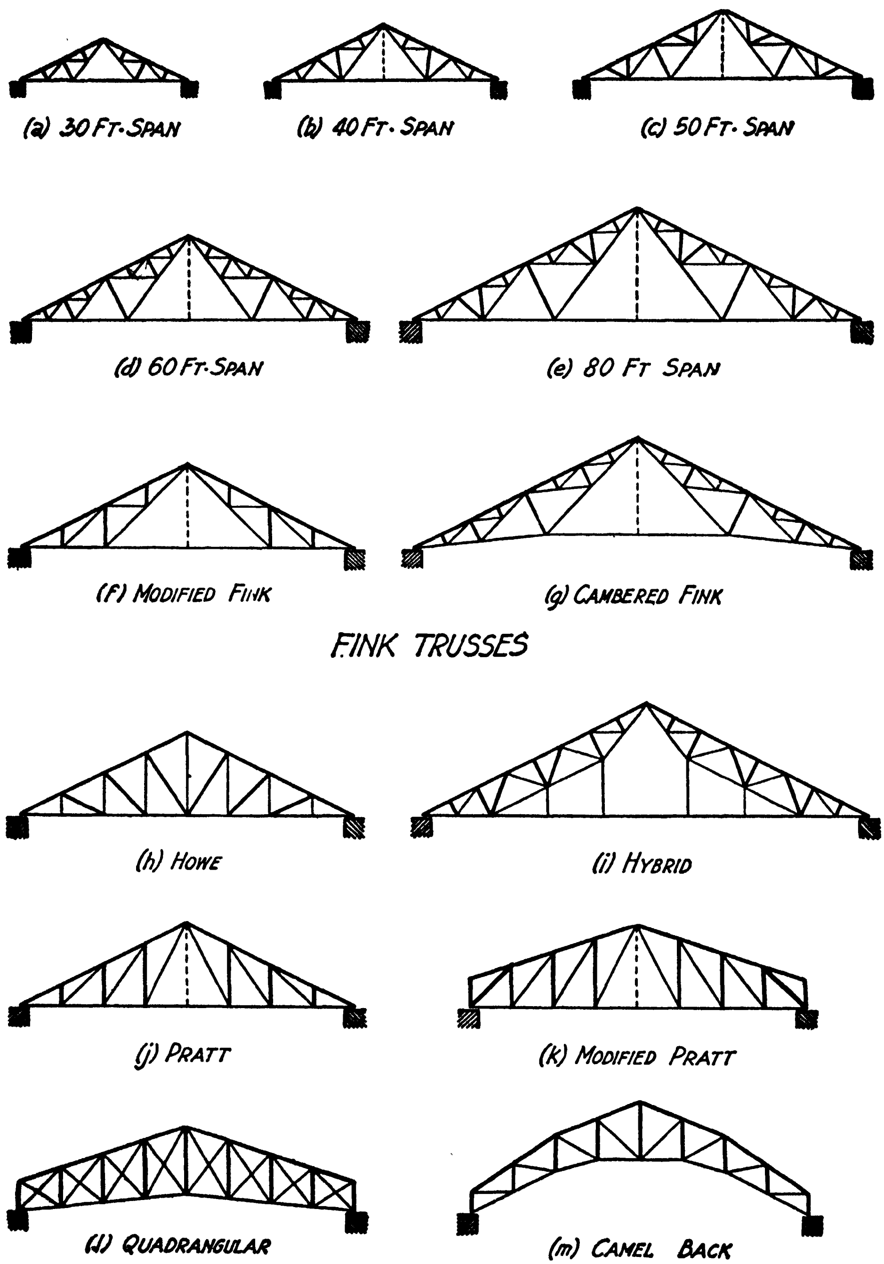


FIG. 5. TYPES OF ROOF TRUSSES.

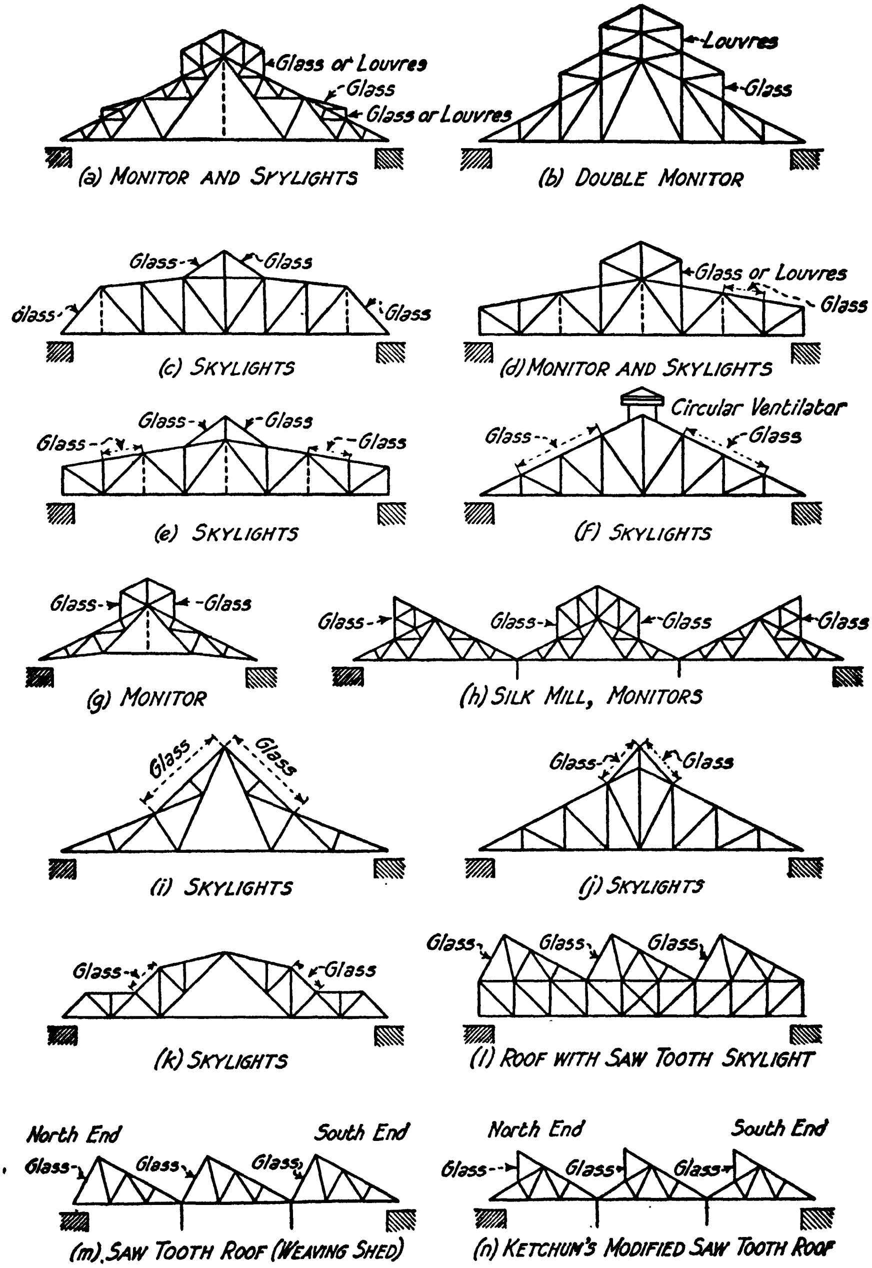


Fig. 6. Roof Trusses Showing Methods of Lighting and Ventilating.

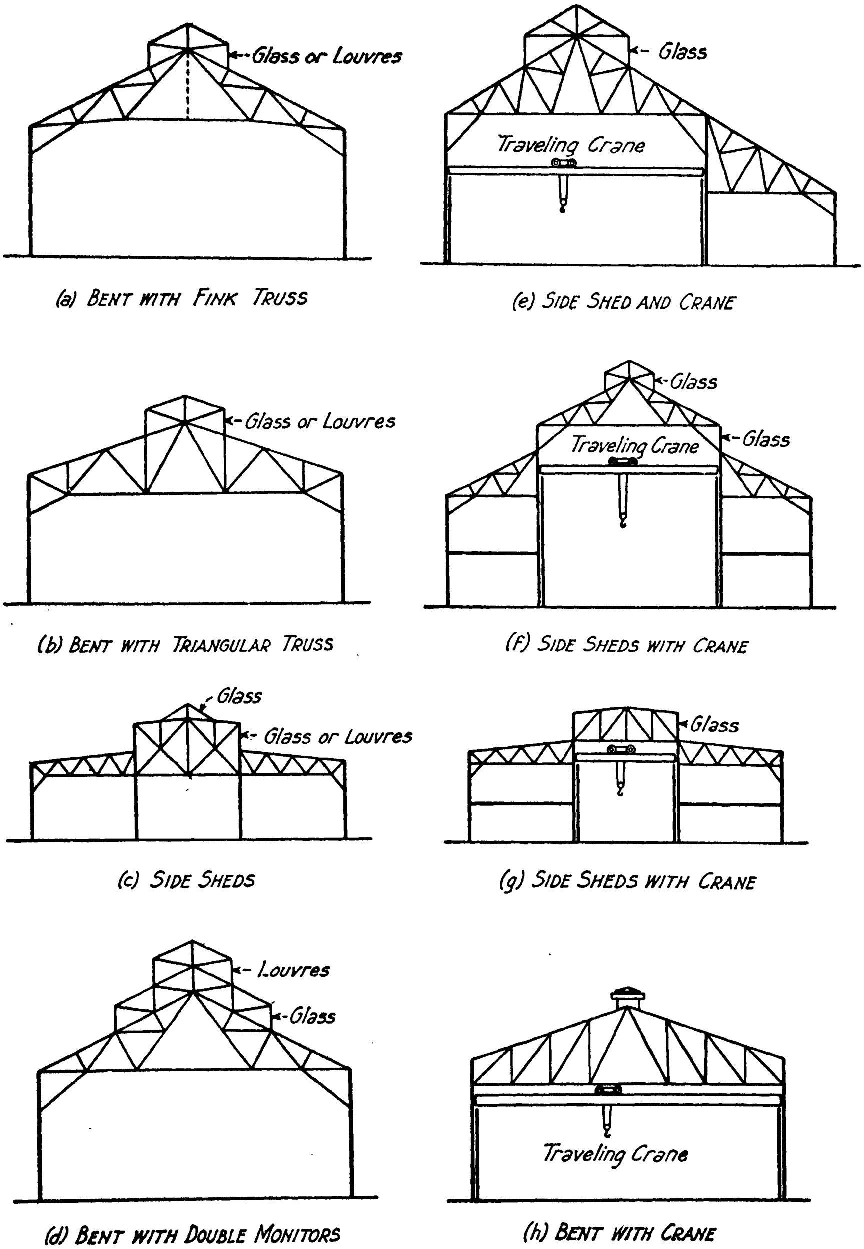
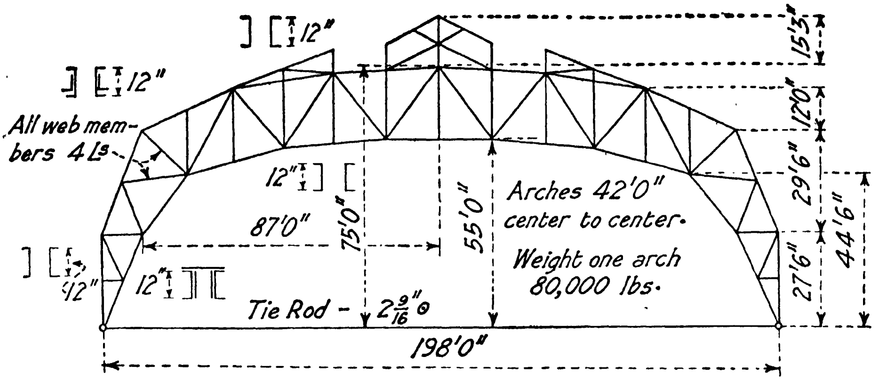
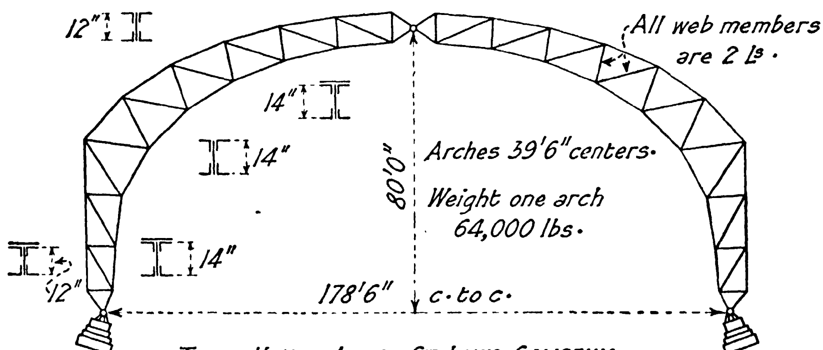


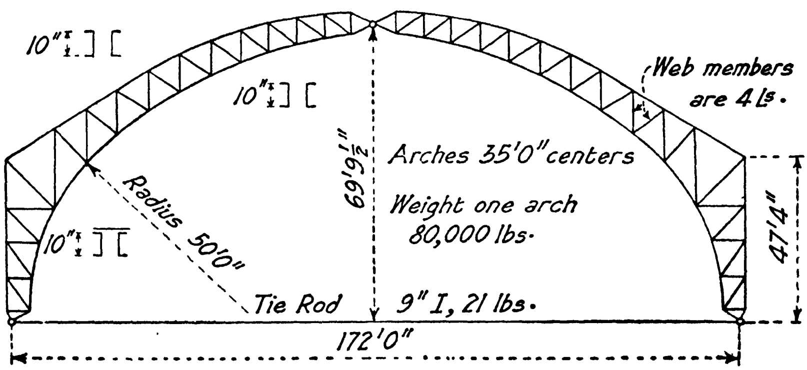
FIG. 7. TYPES OF TRANSVERSE BENTS.



TWO HINGED ARCH, CHICAGO LIVE STOCK PAVILION



THREE HINGED ARCH, ST. LOUIS COLISEUM.



THREE HINGED ARCH, GOVERNMENT BUILDING ST. LOUIS, MO.

FIG. 8. ROOF ARCHES.

and (*g*) are used for shops or buildings where the main part of the building is required to be covered by a crane and side sheds are used for lighter work.

Roof Arches.—Roof arches are used where a large clear floor space is required as in coliseums, exposition buildings and train sheds, Fig. 8. The arches are braced in pairs and carry the roof covering. Arches may have one, two or three hinges, or may be made without hinges. Three-hinged arches are statically determinate structures, while the stresses in all other arches are statically indeterminate. Arches without hinges are used for domes. Three-hinged roof arches have been commonly used in America, although the two-hinged roof arch is more economical and has many advantages. Arches may have a horizontal tie as in the Chicago Stock Pavilion and the Government Building, or the horizontal reactions may be carried by the foundations as in the St. Louis Coliseum, Fig. 8. For the calculation of the stresses in three-hinged and two-hinged roof arches, see the author's "The Design of Steel Mill Buildings."

Pitch of Roof.—The pitch of a roof is given in terms of the center height divided by the span; for example a 60-ft. span truss with ¼ pitch will have a center height of 15 ft. The minimum pitch allowable in a roof will depend upon the character of the roof covering, and upon the kind of sheathing used. For corrugated steel laid directly on purlins, the pitch should preferably be not less than ¼ (6 in. in 12 in.), and the minimum pitch, unless the joints are cemented, not less than ⅕. Slate and tile should not be used on a less slope than ¼ and preferably not less than ⅓. The lap of the slate and tile should be greater for the less pitch. Gravel should never be used on a roof with a greater pitch than about ⅕, and even then the composition is very liable to run. Asphalt is inclined to run and should not be used on a roof with a pitch of more than, say, 2 in. to the foot. If the laps are carefully made and cemented a gravel and tar or asphalt roof may be practically flat; a pitch of ¾ to 1 in. to the foot is, however, usually preferred. Tin may be used on a roof of any slope if the joints are properly soldered. Most of the patent composition roofings give better satisfaction if laid on a roof with a pitch of ⅙ to ¼. Shingles should not be used on a roof with a pitch less than ¼, and preferably the pitch should be ⅓ to ½.

Pitch of Truss.—There is very little difference in the weight of Fink trusses with horizontal bottom chords, in which the top chord has a pitch of ⅕, ¼, or ⅓. The difference in weight is quite noticeable, however, when the lower chord is cambered; the truss with the ⅓ pitch being then more economical than either the ⅕ or the ¼ pitch. Cambering the lower chord of a truss more than, say, 1–40 of the span adds considerable to the weight. For example the computed weights of a 60-ft. Fink truss with a horizontal lower chord, and a 60-ft. Fink truss with a camber of 3 ft. in the lower chord, showed that the cambered truss weighed 40 per cent more for the ¼ pitch and 15 per cent more for the ⅓ pitch, than the truss having the same pitch with horizontal lower chord. It is, however, desirable for appearance sake to put a slight camber in the bottom chords of roof trusses, for the reason that to the eye a horizontal lower chord will appear to sag if viewed from one side.

In deciding on the proper pitch, it should be noted that while the ⅓ pitch gives a better slope and has a less snow load than a roof with ¼ or ⅕ pitch, it has a greater wind load and more roof surface. Taking all things into consideration ¼ pitch is probably the most economical pitch for a roof. A roof with ⅓ pitch is, however, very nearly as economical, and should preferably be used where corrugated steel roofing is used without sheathing, and where the snow load is large.

Spacing of Trusses and Transverse Bents.—The weight of trusses and columns per square foot of area decreases as the spacing increases, while the weight of the purlins and girts per square foot of area increases as the spacing increases. The economic spacing of the trusses is a function of the weight per square foot of floor area of the truss, the purlins, the side girts and the columns, and also of the relative cost of each kind of material. For any given conditions the spacing which makes the sum of these quantities a minimum will be the economic spacing. It is desirable to use simple rolled sections for purlins and girts, and under these conditions the economic spacing will usually be between 16 and 25 ft. The smaller value being about right for spans up to, say, 60 ft., designed for moderate loads, while the greater value is about right for long spans, designed for heavy loads.

Calculations of a series of simple Fink trusses resting on walls and having a uniform span of 60 ft. and different spacings gave the least weight per square foot of horizontal projection of the roof for a spacing of 18 ft., and the least weight of trusses and purlins combined for a spacing of 10 ft. The weight of trusses per square foot was, however, more for the 10-ft. spacing than for the 18-ft. spacing, so that the actual cost of the steel in the roof was a minimum for a spacing of about 16 ft.; the shop cost of the trusses per lb. being several times that of the purlins. Local conditions and requirements usually control the spacing of the trusses so that it is not necessary that we know the economic spacing very definitely.

For long spans the economic spacing can be increased by using rafters supported on heavy purlins, placed at greater distances than would be required if the roof were carried directly by the purlins. This method is frequently used in the design of train sheds and roofs of buildings where plank sheathing is used to support slate or tile coverings, or where the tiles are supported by angle sub-purlins spaced close together as shown in Fig. 13.

Truss Details.—Riveted trusses are commonly used for mill buildings and similar structures. For ordinary loads the chord sections are commonly made of two angles, Fig. 10. For heavy loads the chords may be made of two channels, Fig. 12. Where the purlins are not placed at the panel points the upper chord must be designed for flexure as well as for direct stress. Two angles with a vertical plate make an excellent section where the chord must take both direct and flexural stress. Trusses supported on masonry walls should have one end supported on sliding plates for spans up to 70 ft., for greater lengths of span rollers or a rocker should be used. Shop drawings of a steel roof truss are given in Fig. 10. Details of the end connections of trusses resting on walls and fastened to columns are given in Fig. 11. Details of truss joints are given in Fig. 11. Wherever possible, truss joints should be so designed that the joint will not be eccentric.

Details of Roof Framing.—Roof trusses and transverse bents should be braced transversely with vertical framework and bracing to give the roof framing lateral stability. The bracing may be placed in the center line of the building as in Fig. 12, or at the quarter points as in Fig. 4; long span trusses should be braced at both the center and the quarter points. Details of roof framing giving methods of bracing roof trusses and transverse bents are given in Fig. 4, Fig. 41, and Fig. 42.

Details of a roof truss and roof framing to carry a Ludowici tile roof without sheathing, are shown in Fig. 13. The tiles are carried on sub-purlins, the sub-purlins are supported by rafters, which are in turn supported by the purlins.

Columns.—The common forms of columns used in mill buildings are shown in Fig. 14. For side columns with light loads column (*h*) composed of four angles laced is very satisfactory, while for side columns that take bending and heavy loads column (*g*) composed of four angles and a plate is the most satisfactory column. Columns (*a*), (*b*), (*c*), (*d*), (*e*) and (*k*) are used to carry heavy loads. The I beam and the angle columns are used for end and corner columns, respectively. Details of a four angle laced column and a four angle and plate column are shown in Fig. 15. Details of a heavy column and a light column made of two channels laced are shown in Fig. 16.

CORRUGATED STEEL.—Corrugated steel is rolled to U. S. standard gage. The weights of flat steel and corrugated steel for different gages and thickness are given in Table I. Corrugated siding and roofing is rolled as shown in Fig. 17. The special corrugated steel in (b) Fig. 17 is commonly used for roofing, and the corrugated steel in (c) is used for siding.

The standard stock lengths vary by single feet from 4 ft. to 10 ft. Sheets can be obtained as long as 12 ft., but are special and cost 5 per cent extra and will delay the order.

The purlins for corrugated steel without sheathing should be spaced for a load of 30 lb. per sq. ft. on the roof; and the girts for 25 lb. per sq. ft. on the sides, as given in Fig. 18.

The details of corrugated steel as given in Fig. 19 are standard with the McClintic-Marshall Construction Company and the American Bridge Company.

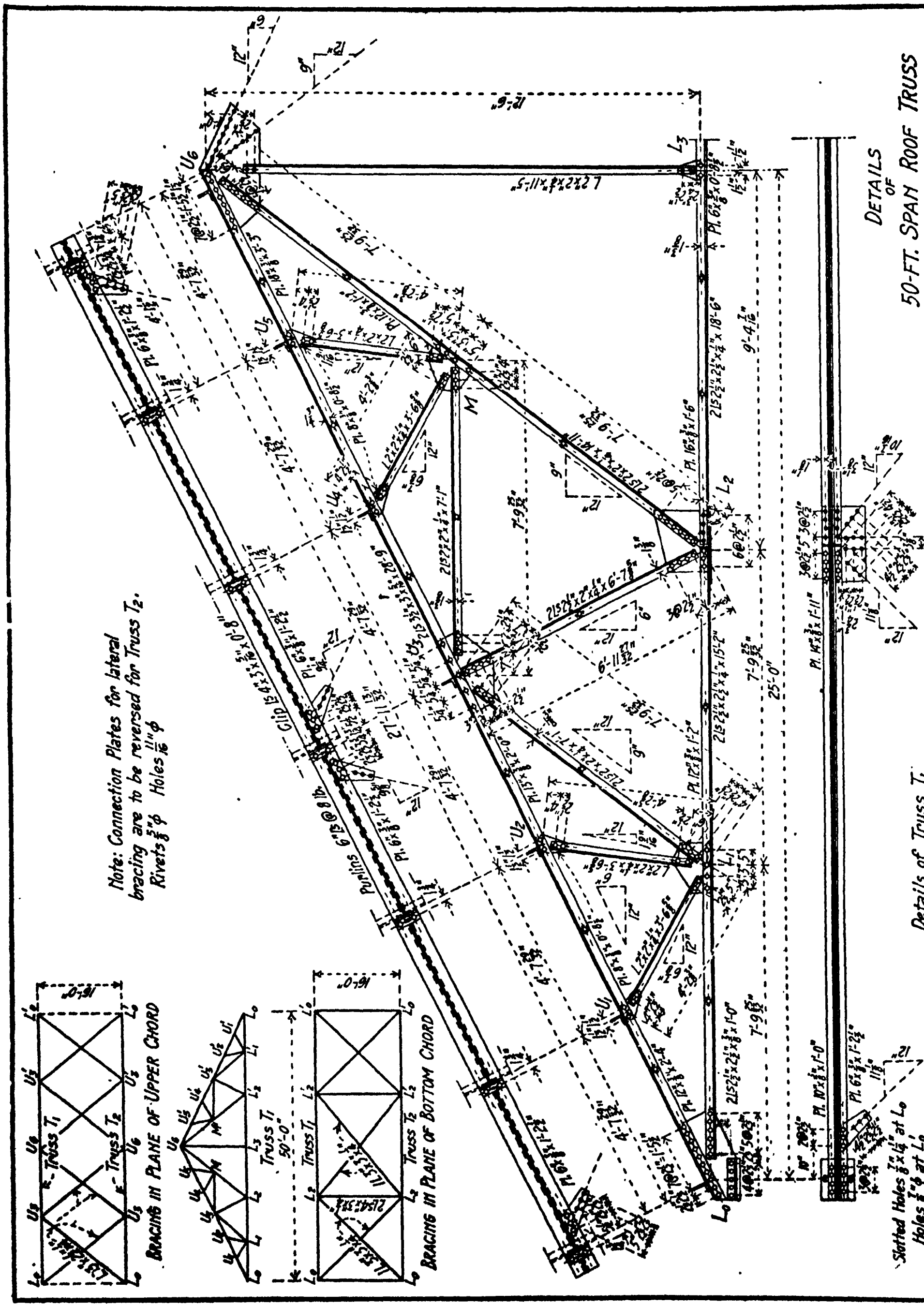


FIG. 10. DETAILS OF A RIVETED STEEL TRUSS.

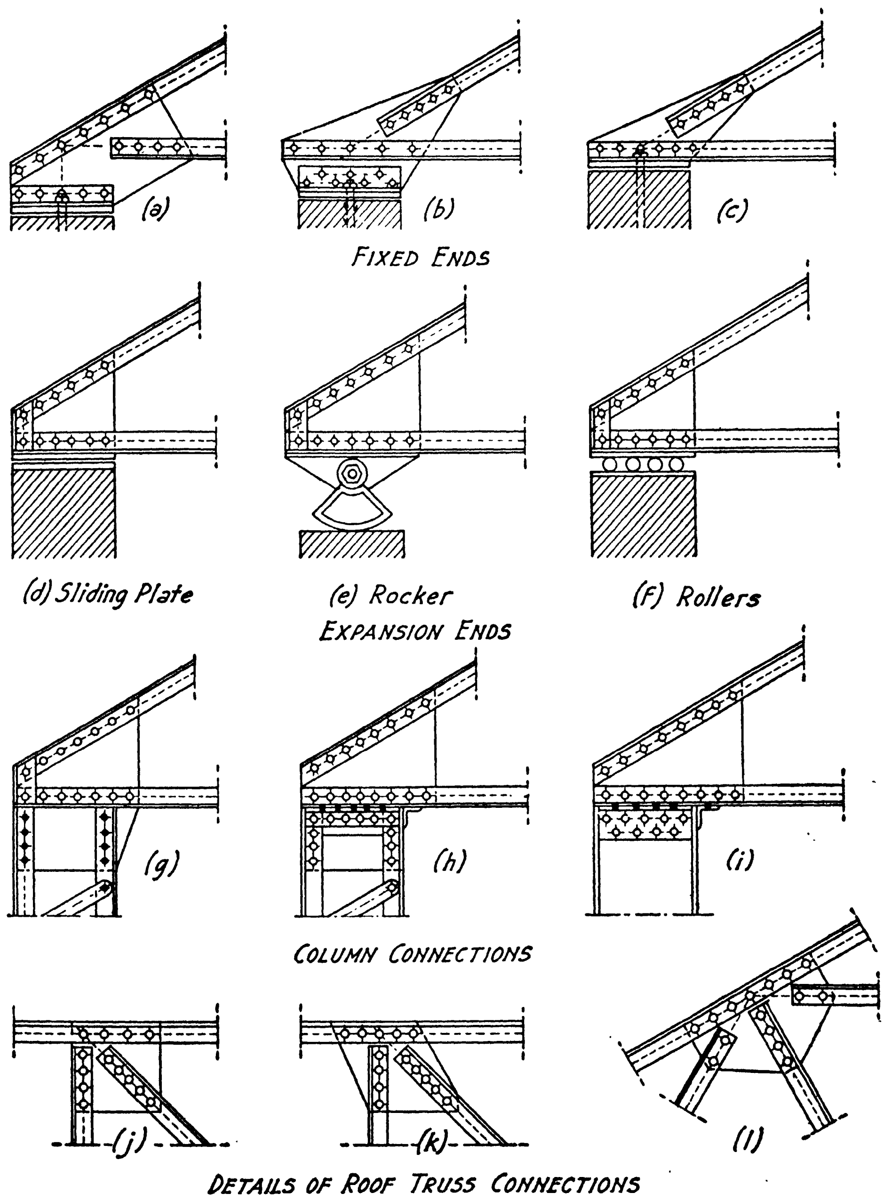


FIG. 11. DETAILS OF TRUSS CONNECTIONS AND JOINTS.

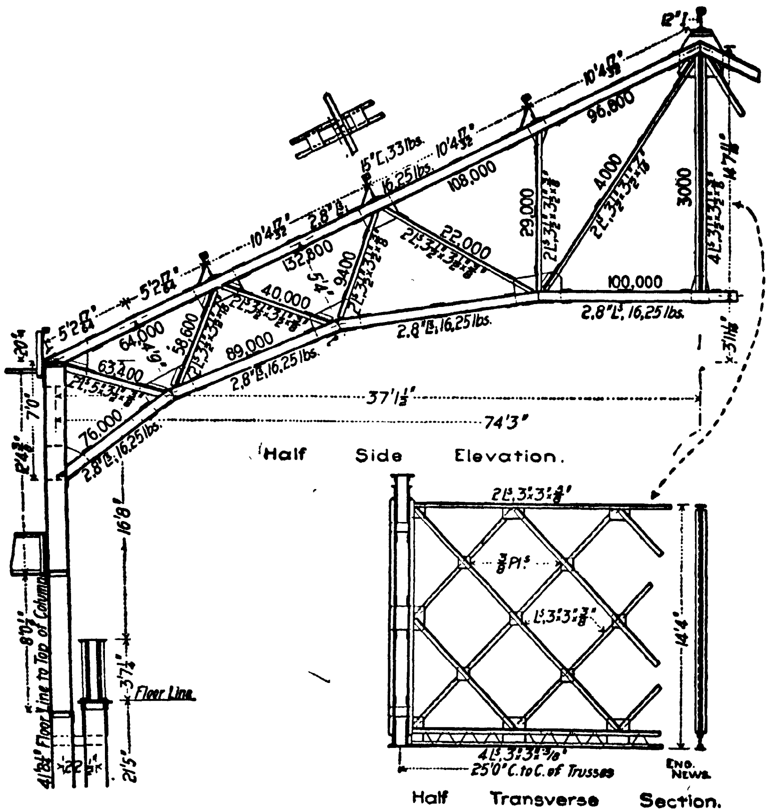


FIG. 12. ROOF TRUSS AND TRANSVERSE BENT SHOWING TRANSVERSE BRACING.

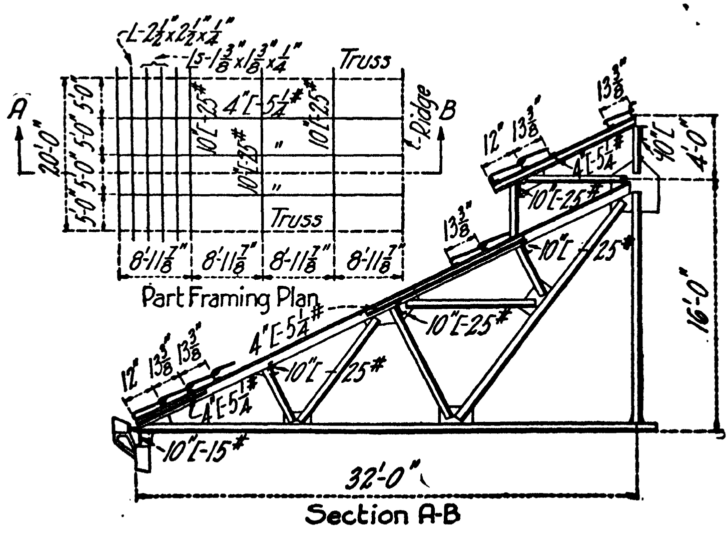


FIG. 13. DETAILS OF A ROOF COVERED WITH LUDOWICI TILE.

Fastenings for Corrugated Sheeting.—Corrugated steel is fastened to purlins and girts usually by the following fasteners.

Straps.—These are made of No. 18 U. S. gage steel, ¾ of an in. wide. These straps pass around the purlins and are riveted to the sheets at both ends by $\frac{3}{16}''$ diameter rivets, ⅜ in. long; or, they may be fastened by bolts. Order one strap and two rivets, or bolts, for each lineal foot of girt or purlin, to which the corrugated steel is to be fastened, and add 20 per cent to the number of rivets for waste, and 10 per cent to the straps or the bolts. One thousand rivets will weigh 6 lb.; one bundle of hoop steel will weigh 50 lb. and contains 400 lineal feet.

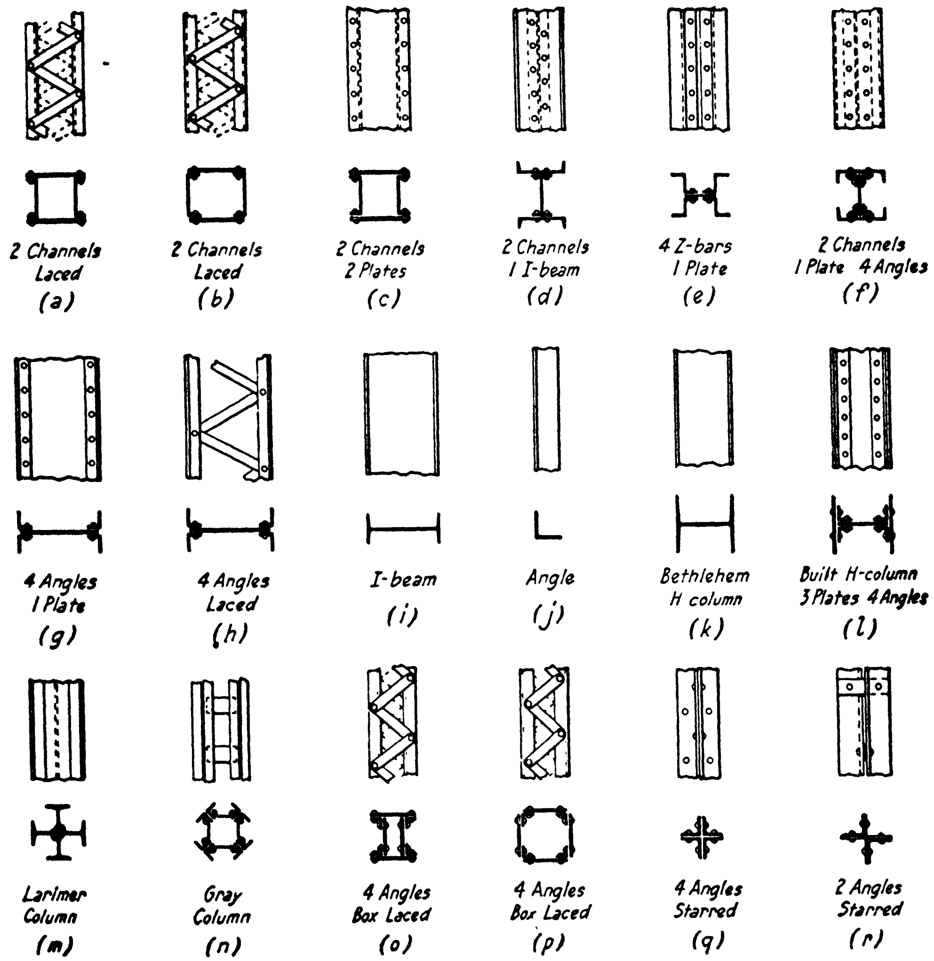
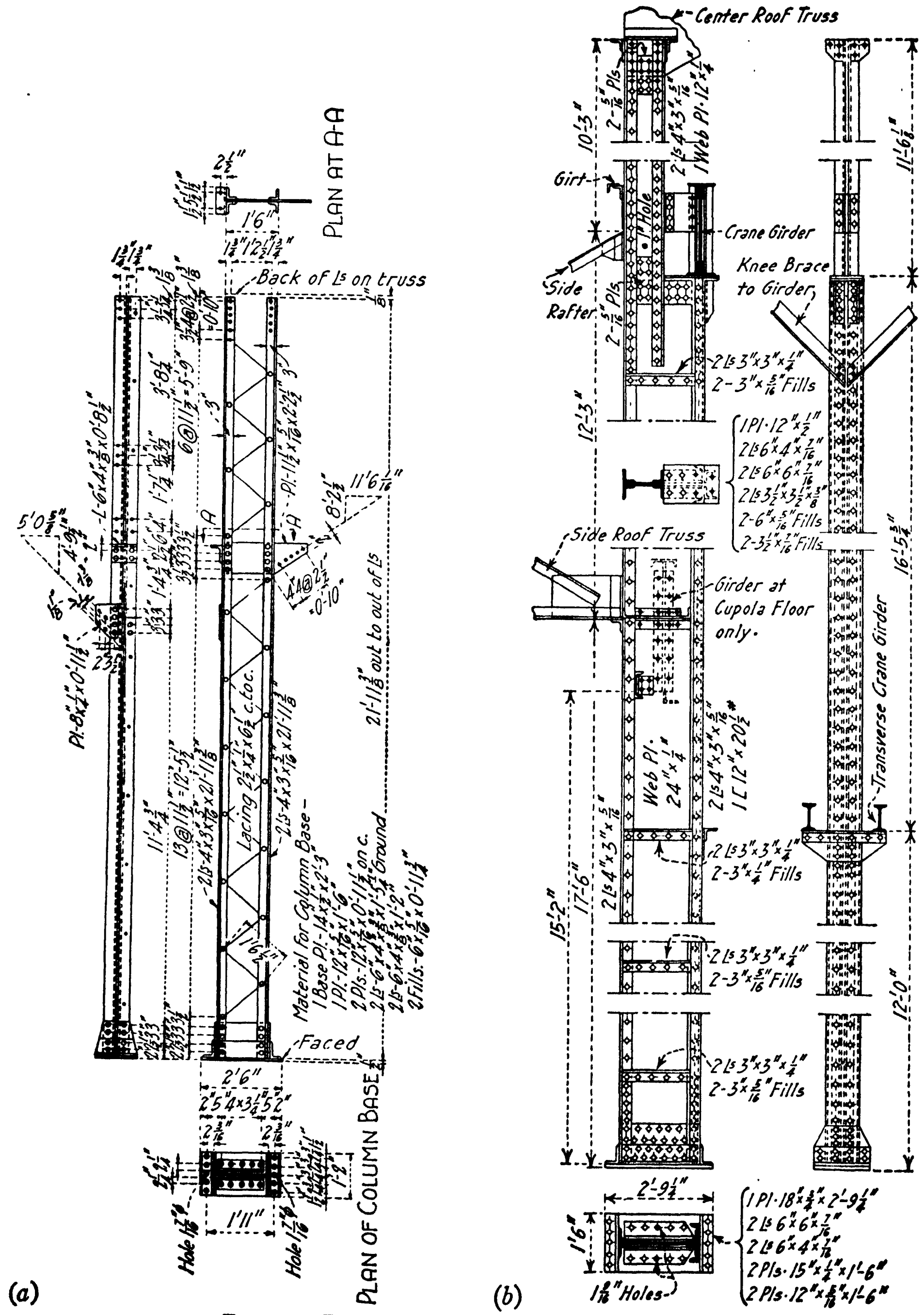


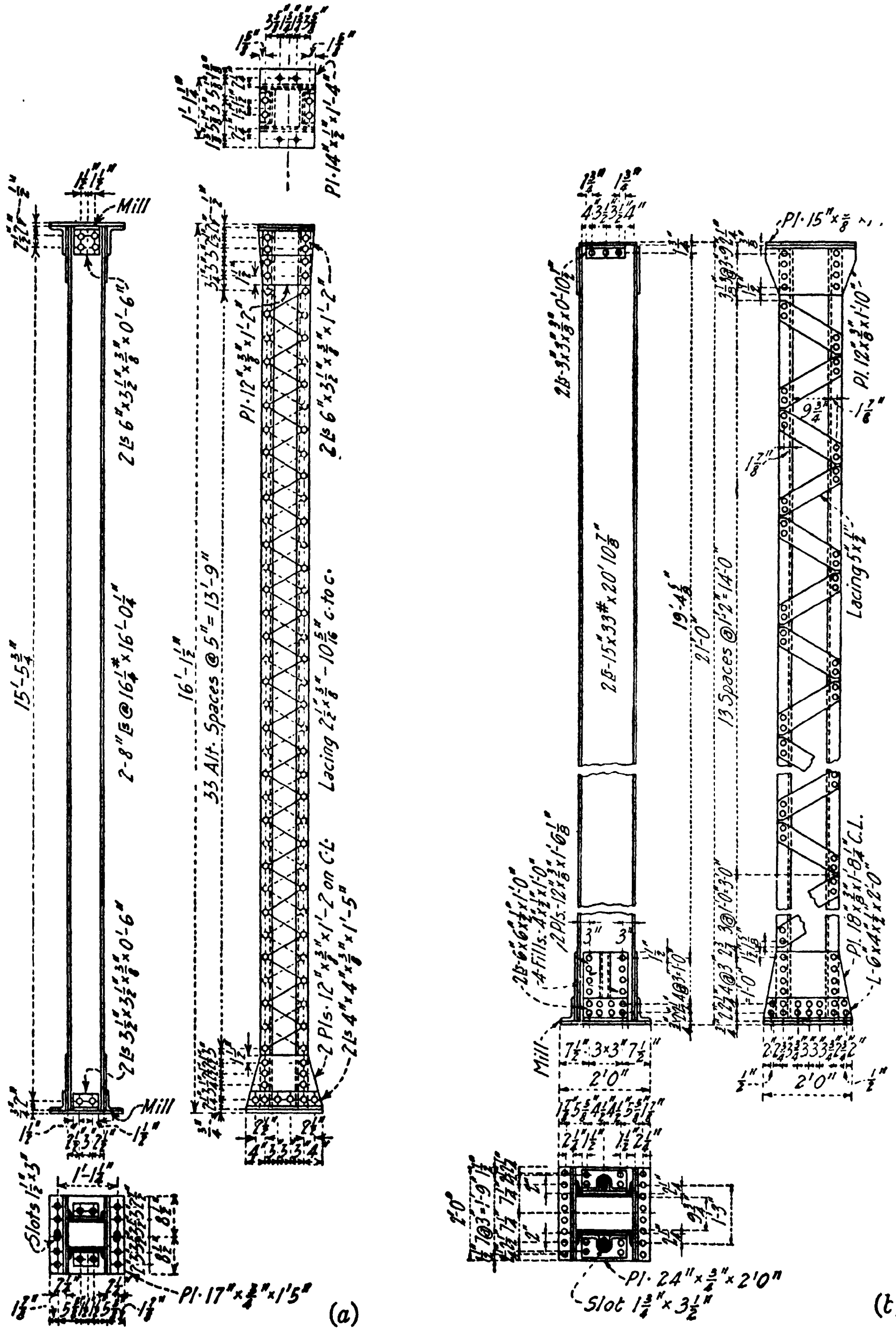
FIG. 14. TYPES OF COLUMNS FOR STEEL MILL BUILDINGS.

Clinch Rivets or Nails.—These are special rivets or nails made of No. 9 Birmingham gage wire, which clinch around the edge of the angle iron or channel and are used for fastening the steel sheathing to steel purlins or girts. They are of the lengths shown on page 24.



(a) (b)

FIG. 15. DETAILS OF MILL BUILDING COLUMNS.



(a) (b)

FIG. 16. DETAILS OF MILL BUILDING COLUMNS.

l foot of purlin or girt to which the corrugated steel is to be waste.

: used for fastening corrugated steel to steel purlins or girts. Clips 1, about 2½ in. long, and are slightly crimped at one end, to go over

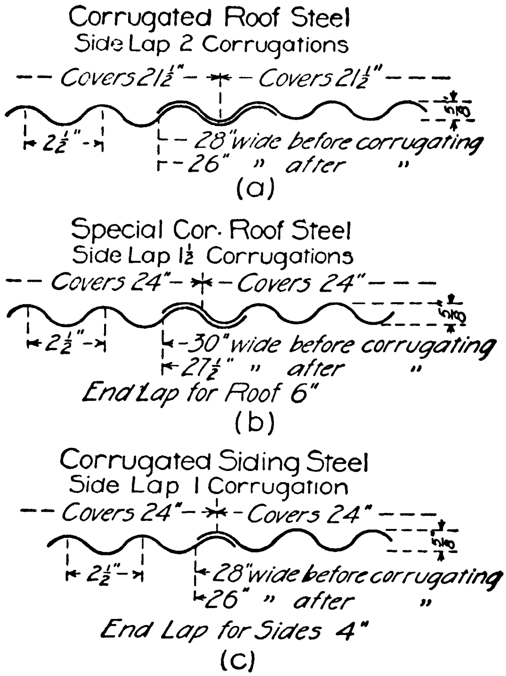


FIG. 17. DETAILS OF CORRUGATED STEEL.

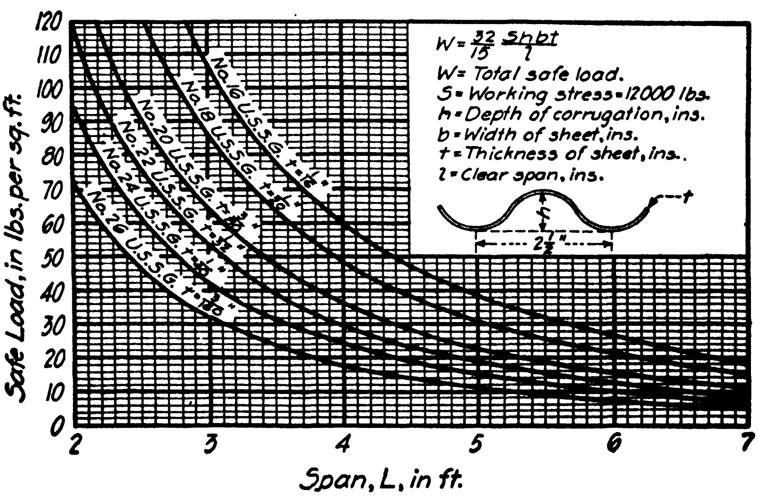
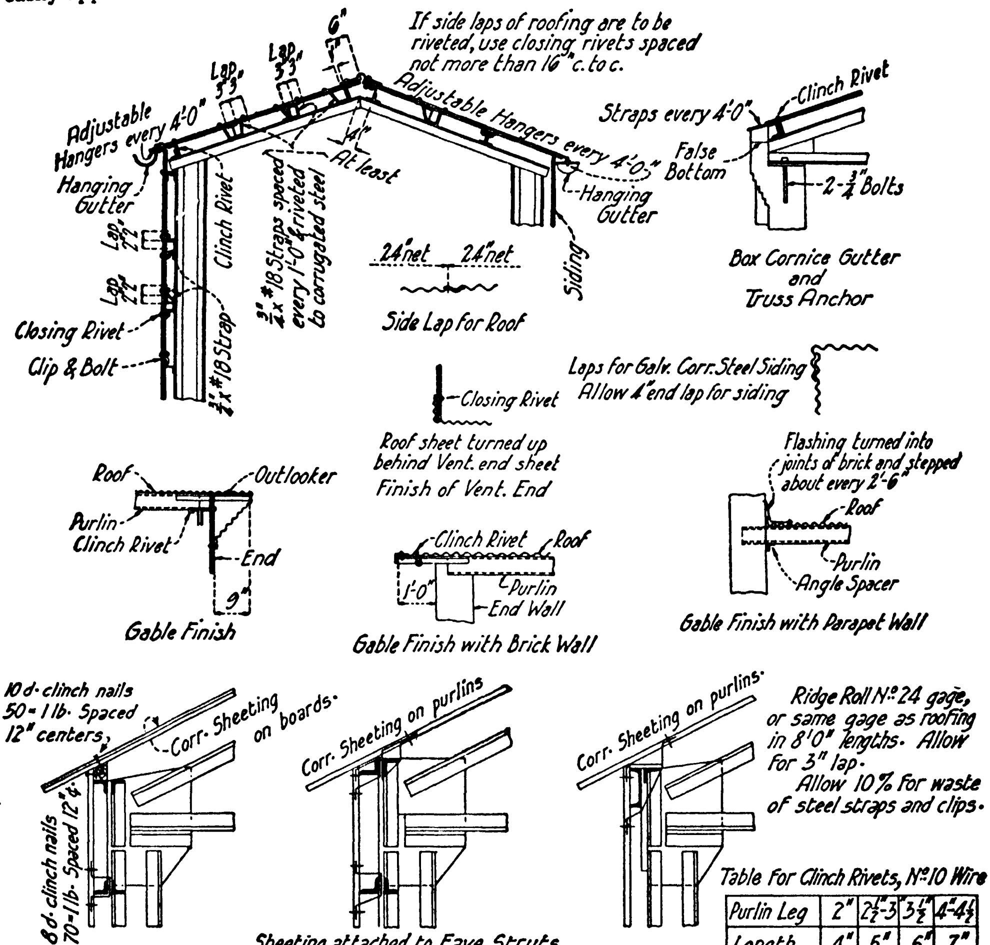


FIG. 18. SAFE LOADS FOR CORRUGATED STEEL.

the flange of the purlin. The bolts are of the same diameter, and have the same head as the clinch rivets, except that they are supplied with threads and nut, and are about 1 in. long. These clips and bolts should not be used excepting in special cases, where the regular fastenings cannot be easily applied.



ROOFING—27½" wide, one edge up and one down, and side lap of 1½ corrugation will cover 24".

Allow 6" end lap for roofs of 6" pitch, 8" for roofs of 4" pitch, 8" for roofs of less than 4" pitch; and lay with Slaters' Cement. If side laps are to be riveted, use closing rivets 12" apart.

SIDING—26" wide, both edges down with side lap of one corrugation, will cover 24". Allow 4" for end lap. Closing rivets in side lap 12" centers.

FLASHING—usually made same gage as siding, can be obtained in following extreme sizes:

No. 16, No. 18 = 48"×144" No. 20 = 48"×120" No. 22, No. 24 = 44½"×120"
No. 26, No. 27 = 40"×120" No. 28 = 40"×96" Order sheets in 8'0" lengths.

Standard Corrugated Roofing and Siding can be obtained 48"×132" varying by 6". Corrugations approximately 2½"×½".

Table for Clinch Rivets, No. 10 Wire

Purlin Leg	2"	2½"-3"	3½"	4"-4½"
Length	4"	5"	6"	7"
No. per lb.	48	38	33	27

Spaced 6" apart. 10% for waste

Closing Rivets

Diameter	3/16"	3/16"	3/16"	3/16"
Length	3/8"	1/2"	5/8"	3/4"
No. per lb.	200	166	142	125

Allow 20% for waste

FIG. 19. STANDARD DETAILS FOR CORRUGATED STEEL.

TABLE OF CLINCH NAILS.

L Purlin leg	3″	4″	5″	6″	7″
Length	5″	6″	7″	8″	9″
No. per lb.	32	29	23	21	18
L Purlin leg	3″	4″	5″	6″	7″
Length	6″	7″ or 8″	9″	10″	11″
No. per lb.	29	21	18	16	14

In cases where flashing, cornice work, and several thicknesses of metal are to be fastened at one point, rivets or bolts, other than standard lengths given will be needed. Closing rivets ½ in. long and bolts 1½ in. long will usually answer in these cases.

If side laps of corrugated steel are to be riveted, rivets should be ordered, one for each lineal foot of side lap, plus 20 per cent for waste.

If corrugated steel is to be fastened to wooden purlins or timber sheathing, order 8d barbed nails for roofing and for siding. These nails should be spaced one foot apart, for both end and side laps; add 20 per cent for waste. Ninety-six 8d barbed nails weigh 1 lb.

Corrugated steel for roofing should be laid with two corrugations side lap if standard or 1½ corrugations side lap if special, and 6 in. end lap. Corrugated steel for siding should have one corrugation side lap and 4 in. end lap.

Louvres.—Weights of Shiffier louvres of black iron or steel are as follows:

Gage No.	Weight per Square Feet.
20	2.7 lb.
22	2.0 lb.

The weight is obtained from Fig. 20, as follows:

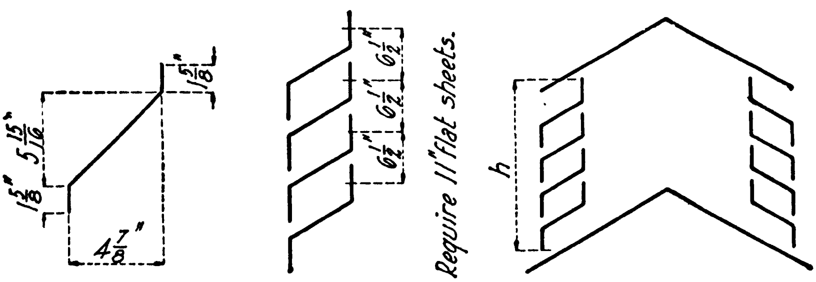


FIG. 20. LOUVRES.

Louvres are estimated in square feet = 2h × length.

To get weight multiply area by (1.7 × weight per sq. ft. of flat of material used).

Ridge Roll.—Ridge roll is ordinarily of same gage as roofing and black or galvanized to correspond with same. Ridge roll is usually made from an 18 in. flat sheet.

WEIGHT OF RIDGE ROLL.

Gage No.	Weight, lb. per lineal ft.
20	2.4 Black Iron or Steel.
22	2.0 Black Iron or Steel.
24	1.6 Black Iron or Steel.

TABLE I.

Corrugated Sheets. American Sheet and Tin Plate Company Standard.

Description of Corrugated Sheets						Areas of Corrugated Sheets						
Corrugations				Width, Inches		Length of Sheet, Inches	Sq. Ft. in 1 Sheet			Sheets in 100 Sq. Ft.		
Width, Inches		Depth, Approx. Inches	Number per Sheet	Full Sheet	Covers Approx.		Corrugations			Corrugations		
Nominal	Actual						5″	3″, 2½″, 2″	1¼″, ⅝″	5″	3″, 2½″, 2″	1¼″, ⅝″
5	4⅔	⅞	6	28	24	60	11.67	10.83	10.42	8.57	9.23	9.60
3	2 8/9	⅝	9	26	24	72	14.00	13.00	12.50	7.14	7.69	8.00
2½	2⅗	½	10	26	24	84	16.33	15.17	14.58	6.12	6.59	6.86
2	2 4/11	½	11	26	24	96	18.67	17.33	16.67	5.36	5.77	6.00
1¼	1¼	⅜	20	25	24	108	21.00	19.50	18.75	4.76	5.13	5.33
⅝	25/26	5/16	26	25	24	120	23.33	21.67	20.83	4.29	4.62	4.80
Standard lengths 5, 6, 7, 8, 9 and 10 feet. Maximum length, 12 feet for 5″ to 1¼″ corrugation.						144	28.00	26.00	25.00	3.57	3.85	4.00

Corrugated Sheets.—Painted.
Weights in Pounds per 100 Square Feet.

Nom. Corrug. Inches	Thickness, U. S. Standard Gage and Decimals of an Inch												
	12	14	16	18	20	21	22	23	24	25	26	27	28
	.109	.078	.063	.050	.038	.034	.031	.028	.025	.022	.019	.017	.016
5	......	339	271	217	163	150	136	123	110	96	83	76	68
3	......	......	271	217	163	150	136	123	110	96	83	76	68
2½	474	339	271	217	163	150	136	123	110	96	83	76	68
2	......	......	271	217	163	150	136	123	110	96	83	76	68
1¼	......	......	......	......	170	156	142	128	114	100	86	79	72
⅝	......	......	......	......	......	......	......	......	114	100	86	79	72

Corrugated Sheets.—Galvanized.
Weights in Pounds per 100 Square Feet.

Nom. Corrug. Inches	Thickness, U. S. Standard Gage and Decimals of an Inch												
	12	14	16	18	20	21	22	23	24	25	26	27	28
	.109	.078	.063	.050	.038	.034	.031	.028	.025	.022	.019	.017	.016
5	......	354	286	232	178	165	151	138	124	111	98	91	85
3	......	......	286	232	178	165	151	138	124	111	98	91	85
2½	488	354	286	232	178	165	151	138	124	111	98	91	85
2	......	......	286	232	178	165	151	138	124	111	98	91	85
1¼	......	......	......	......	185	......	157	......	129	......	101	94	87
⅝	......	......	......	......	......	......	......	......	129	......	101	94	87

The weights per 100 square feet given in preceding tables do not include allowances for end or side laps. The following table gives the approximate number of square feet of sheeting necessary to cover an area of 100 square feet and is based on sheets of standard width, 96 inches long. If longer or shorter sheets are used, the number of square feet required will vary accordingly.

Square Feet of Corrugated Sheets to Cover 100 Square Feet.

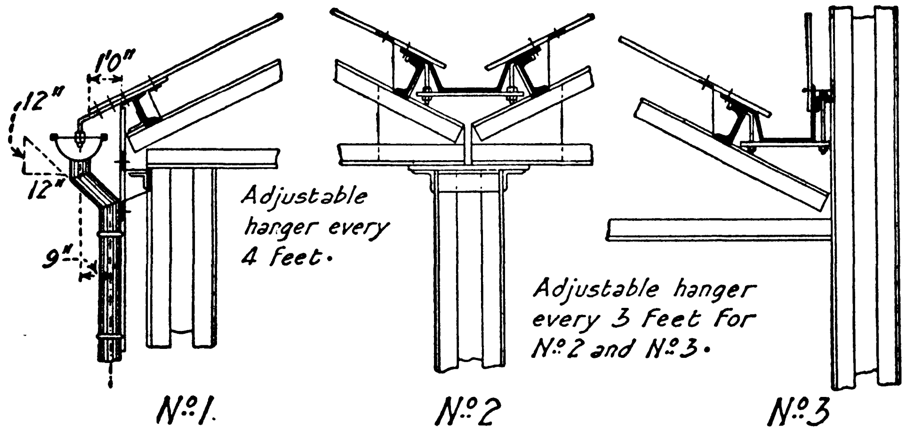
Side Lap	End Lap, Inches					
	1	2	3	4	5	6
1 Corrugation..............	110	111	112	113	114	115
1½ "	116	117	118	119	120	121
2 "	123	124	125	126	127	128

Gutters.—Eave or valley gutters should always be galvanized. Valley gutters should be No. 20 gage. Eave gutters and conductors should be No. 22 gage. Gutters should be sloped not less than 1 in. in 15 ft.

WEIGHTS OF EAVE GUTTERS AND CONDUCTORS OF GALV. IRON OR STEEL.

Span of Roof.	Size of Gutter.	Wt. per ft.	Size and Spacing of Conductor.	Wt. per lin. ft. No. 22.
up to 50′	6″, No. 22	1.8 lb.	4 in. every 40′ 0″	1.5 lb.
50′ to 70′	7″, No. 22	1.9 lb.	5 in. every 40′ 0″	2.1 lb.
70′ to 100′	8″, No. 22	2.1 lb.	5 in. every 40′ 0″	2.3 lb.

Details of conductors and downspouts are given in Fig. 21.



Type	Area Drained Sq. Ft.	Size of Gutter	Conductors Diam. Ins.	Conductors Spaced Ft.
No. 1	0 to 1200	6″	4	40
	1200 to 1800	7″	5	40
	1800 to 2400	8″	5	40
No. 2 and No. 3	0 to 2400	4″x8″	5	40
	2400 to 3600	5″x8″	6	40
	3600 to 4800	5″x10″	6	40

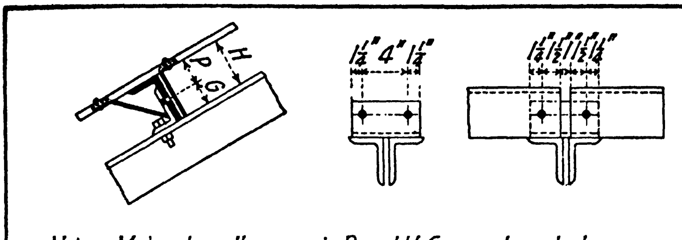
Eave and Valley Gutters usually No. 20 or same gage as roofing.
Slope one inch in fifteen feet.
Order in 8 feet lengths.
Condutcors usually No. 22 or same gage as siding.

FIG. 21. DETAILS OF CONDUCTORS AND DOWNSPOUTS. AMERICAN BRIDGE COMPANY.

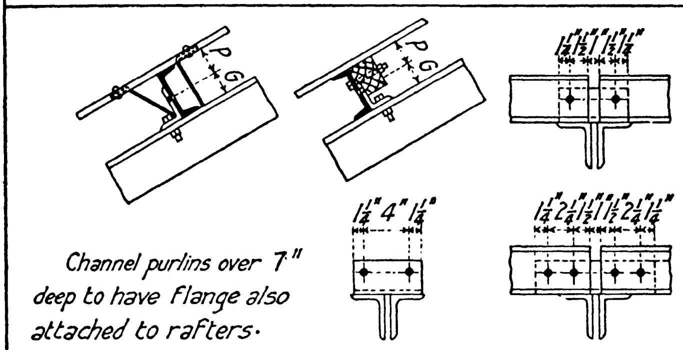
Purlins.—Details of connections for purlins used for a corrugated steel roof are given in Fig. 22.

Cornice.—For details of cornice see the author's "The Design of Steel Mill Buildings."

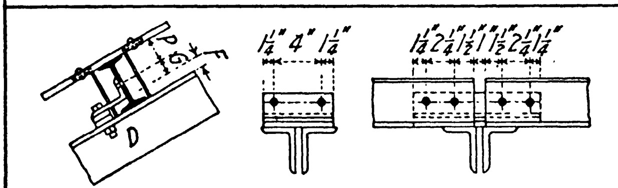
ROOF COVERINGS.—Mill buildings are covered with corrugated steel supported directly on the purlins; by slate, tile or cement tile supported by sub-purlins; or by corrugated steel, slate, tile, cement tile, shingles, gravel or other composition roof, or some one of the various patented roofings supported on sheathing. The sheathing is commonly made of a single thickness



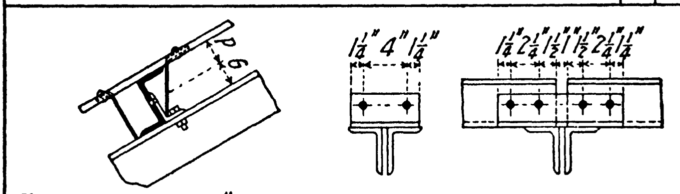
ANGLE PURLINS			
Leg H	Clip Angle.	P	G
2"	1¾×1¾"	1"	1"
2½	2×2	1⅜	1⅛
3	2½×2½ or 2	1½	1½
3½	3×2½	1¾	1¾
4	3×2½	2	2
5	3½×2½	2½	2½



CHANNEL PURLINS			
Purlin	Clip Angle	P	G
4"	3"×2½"	2"	2"
5	3½×2½	2½	2½
6	4×3	3	3
7	4×3	4	3
8	4×3	5	3
9	5×3½	5½	3½
10	5×3½	6½	3½



I BEAM PURLINS					
Purlin	Clip Angle	P	G	F	D
4"	3½"×2½"	2"	1¼"	¾"	2½"
5	4×3	2½	1¾	¾	2⅞
6	4×3	3	2	1	3
7	5×3½	4	2	1	3⅜



Z BAR PURLINS			
Purlin	Clip Angle	P	G
3"	2½"×2½" or 2"	1½"	1½"
4	3×2½	2	2
5	3½×2½	2½	2½
6	4×3	3	3

Purlins or girts should extend, where possible, over two or more bays with joints staggered. Where purlins act as struts, use clip with four holes.

Where purlins are punched for nailing strips, space holes about 3'0" apart. Bolt purlins to clips and clips to trusses unless otherwise specified.

FIG. 22. DETAILS OF PURLINS FOR CORRUGATED STEEL ROOF. AMERICAN BRIDGE COMPANY.

of planks, 1 to 3 inches thick. The planks are sometimes laid in two thicknesses with a layer of lime mortar between the layers as a protection against fire. In fireproof buildings the sheathing is commonly made of reinforced concrete. Concrete slabs are sometimes used for a roof covering, being in that case supported directly by the purlins, and sometimes as a sheathing for a slate or tile roof.

The roofs of smelters, foundries, steel mills, mine structures and similar structures are commonly covered with corrugated steel. Where the buildings are to be heated or where a more substantial roof covering is desired slate, tile, tin or a good grade of composition roofing is used, or the roof is made of reinforced concrete. For very cheap and for temporary roofs a cheap composition roofing is commonly used. The following coverings will be described in the order given; corrugated steel, slate, tile, tin, and tar and gravel. A slate roof on reinforced concrete sheathing is shown in Fig. 57 and in Fig. 59.

CORRUGATED STEEL ROOFING.—Corrugated steel roofing is laid on plank sheathing or is supported directly on the purlins. Corrugated steel roofing should be kept well painted with a good paint. Where the roofing is exposed to the action of corrosive gases as in the roof of a smelter reducing sulphur ores, ordinary red lead or iron oxide paint is practically worthless as a protective coating; better results being obtained with graphite and asphalt paints. Tar paint, made by mixing tar, Portland cement and kerosene in the proportions of 16 parts of tar, 4 parts of Portland cement, and 3 parts of kerosene, by volume, is an excellent protection against corrosive gases in smelters and similar structures. Galvanized corrugated steel is quite extensively used. To prevent the condensation of vapor on the inside of the metal roof, corrugated steel roofing should be laid on sheathing or should have anti-condensation lining.

Corrugated steel sheets covered with an asbestos preparation can now be obtained on the market.

Anti-Condensation Lining.—Anti-condensation lining, shown in Fig. 23, consists of asbestos felt supported on wire netting that is stretched tight and supported by the purlins. Anti-condensation lining is put on according to two systems.

Berlin System, (5) Fig. 23.—(1) Lay galvanized wire netting, No. 19, 2-in. mesh, transversely to the purlins with edges about 1½ in. apart so that when laced together with No. 20 brass wire the netting will be stretched smooth and tight. When the purlins are spaced more than 4 ft. apart stretch No. 9 galvanized wire across the purlins about 2 ft. centers to hold up the netting.

(2) On the top of the wire netting place a layer of asbestos paper weighing 14 lb. per square of 100 sq. ft., and on this place a layer of asbestos paper weighing 6 lb. per square. All holes in the paper must be patched when laid.

(3) On top of the asbestos paper lay two thicknesses of Neponset building paper.

Note.—The asbestos and building paper should lap 3 in. and break joints 12 in. The corrugated steel is fastened with the usual connections. Use tin washers on corrugated steel bolts where there is danger of breaking or tearing the lining.

Wire netting, No. 19 gage, 2-in. mesh comes in bundles 6 ft. wide and 150 ft. long, containing 900 sq. ft. Asbestos comes in rolls 36 in. wide and is sold by the pound. No. 20 brass wire is bought by the pound, 272 lineal ft. weigh one pound. Neponset building paper comes in rolls 36 in. wide and 250 ft. or 500 ft. long. Do not cut a roll. Add 10 per cent for laps of asbestos and building paper.

Minneapolis System, (6) Fig. 23.—(1) Lay wire netting, No. 19, 2-in. mesh, transversely to the purlins, with edges 1½ in. apart, so that when laced together with No. 20 brass wire the netting will be stretched smooth and tight.

(2) On the top of the netting lay asbestos paper weighing 30 lb. to the square of 100 sq. ft., allowing 3 in. for laps. For important work lay one or two thicknesses of building paper on top of the asbestos.

(3) Lay the corrugated steel and fasten to purlins in the usual manner.

Note.—If wood purlins are used the wire netting may be fastened to the nailing strips with ⅜ in. staples. Where the purlins are more than 2 ft. 6 in. centers place a line of $\frac{3}{16}$ in. bolts between purlins, about 2 ft. centers, with washers 1 in. × 4 in. × ⅛ in. to prevent netting from sagging.

SLATE ROOFING.—Roofing slates are usually made from ⅛ to ¼ inches thick; $\frac{3}{16}$ inch being a very common thickness. Slates vary in size from 6 in. × 12 in. to 24 in. × 44 in.; the sizes varying from 6 in. × 12 in. to 12 in. × 18 in. being the most common.

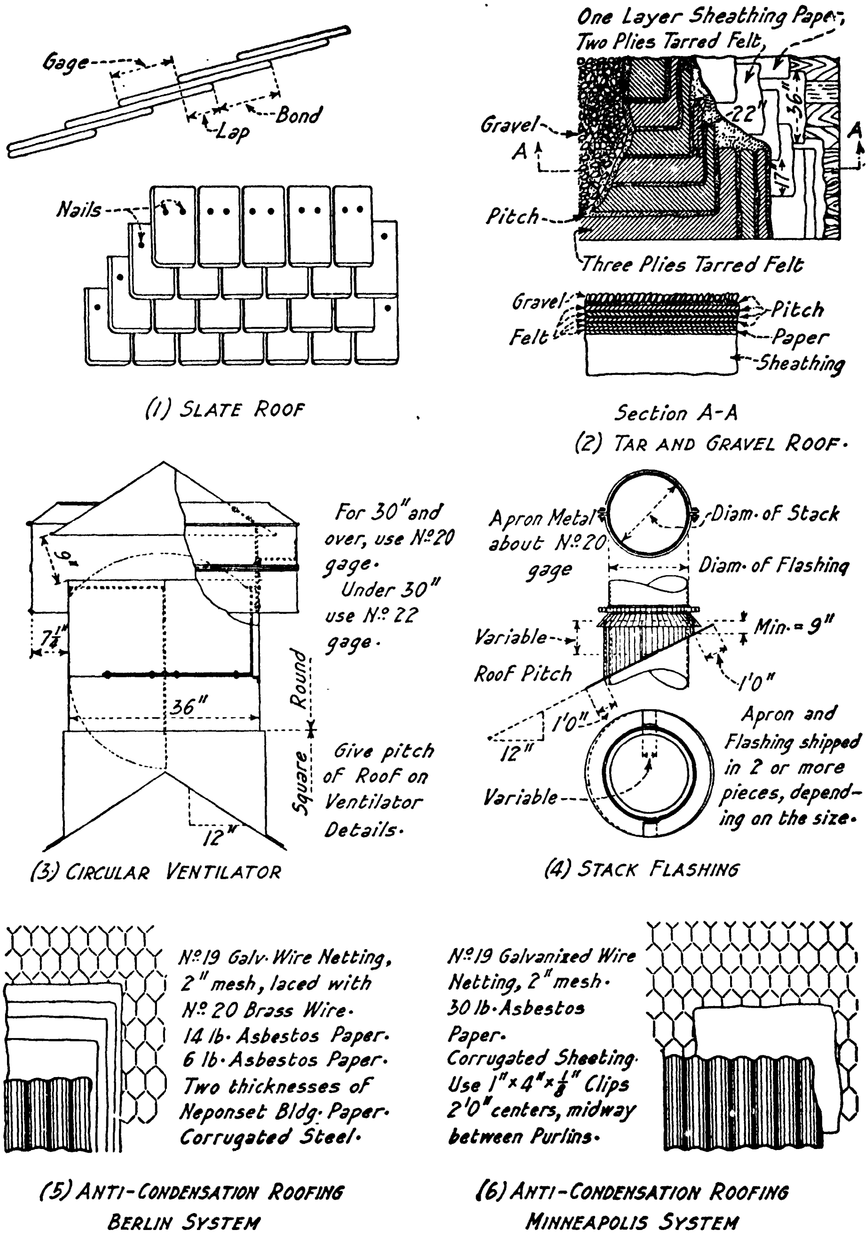


FIG. 23. DETAILS OF ROOFING, VENTILATORS AND ANTI-CONDENSATION LINING.

Slates are laid like shingles as shown in Fig. 23. The lap most commonly used is 3 inches; where less than the minimum pitch of $\frac{1}{4}$ is used the lap should be increased. The number of slates of different sizes required for one square of 100 sq. ft. of roof for a 3-in. lap are given in Table II. The weight of slates of the various lengths and thicknesses required for one square of roofing, using a 3-in. lap is given in Table III. The weight of slate is about 174 lb. per cu. ft. The weight of slate per superficial sq. ft. for different thicknesses is given in Table IV.

TABLE II.

NUMBER OF ROOFING SLATES REQUIRED TO LAY ONE SQUARE OF ROOF WITH 3-IN. LAP.

Size in Inches.	No. of Slate in Square.	Size in Inches.	No. of Slate in Square.	Size in Inches.	No. of Slate in Square.
6 × 12	533	8 × 16	277	12 × 20	141
7 × 12	457	9 × 16	246	14 × 20	121
8 × 12	400	10 × 16	221	11 × 22	137
9 × 12	355	12 × 16	184	12 × 22	126
10 × 12	320	9 × 18	213	14 × 22	108
12 × 12	266	10 × 18	192	12 × 24	114
7 × 14	374	11 × 18	174	14 × 24	98
8 × 14	327	12 × 18	160	16 × 24	86
9 × 14	291	14 × 18	137	14 × 26	89
10 × 14	261	10 × 20	169	16 × 26	78
12 × 14	218	11 × 20	154	....	....

TABLE III.

THE WEIGHT OF SLATE REQUIRED FOR ONE SQUARE OF ROOF.

Length in Inches.	Weight in pounds, per square, for the thickness.							
	$\frac{1}{8}''$	$\frac{3}{16}''$	$\frac{1}{4}''$	$\frac{3}{8}''$	$\frac{1}{2}''$	$\frac{5}{8}''$	$\frac{3}{4}''$	1″
12	483	724	967	1450	1936	2419	2902	3872
14	460	688	920	1370	1842	2301	2760	3683
16	445	667	890	1336	1784	2229	2670	3567
18	434	650	869	1303	1740	2174	2607	3480
20	425	637	851	1276	1704	2129	2553	3408
22	418	626	836	1254	1675	2093	2508	3350
24	412	617	825	1238	1653	2066	2478	3306
26	407	610	815	1222	1631	2039	2445	3263

TABLE IV.

WEIGHT OF SLATE PER SQUARE FOOT.

Thickness—in........	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	1
Weight—lb.........	1.81	2.71	3.62	5.43	7.25	9.06	10.87	14.5

The minimum pitch recommended for a slate roof is $\frac{1}{4}$; but even with steeper slopes the rain and snow may be driven under the edges of the slates by the wind. This can be prevented by laying the slates in slater's cement. Cemented joints should always be used around eaves, ridges and chimneys.

Slates are commonly laid on plank sheathing. The sheathing should be strong enough to prevent deflections that will break the slate, and should be tongued or grooved, or shiplapped, and dressed on the upper surface. Concrete sheathing reinforced with wire mesh, expanded metal or rods is now being used quite extensively for slate and tile roofs, and makes a fireproof roof, see

Fig. 46. Tar roofing felt laid between the slates and the sheathing assists materially in making the roof waterproof, and prevents breakage when the roof is walked on. The use of rubber-soled shoes by the workmen will materially reduce the breakage caused by walking on the roof. Roofing slates may also be supported directly on sub-purlins. The details of this method are practically the same as for tile roofing, which see.

When roofing slates are laid on sheathing they are fastened by two nails, one in each upper corner, Fig. 23. When supported directly on sub-purlins the slates are fastened by copper or composition wire. Galvanized and tinned steel nails, copper, composition and zinc slate roofing nails are used. Where the roof is to be exposed to corrosive gases copper, composition or zinc nails should be used.

TILE ROOFING.—Baked clay or terra-cotta roofing tiles are made in many forms and sizes. Plain roofing tiles are usually 10½ in. long, 6¼ in. wide and ⅝ in. thick; weigh from 2 to 2½ lb. each and lay one-half to the weather. There are many other forms of tile among which book tile, Spanish tile, pan tile and Ludowici tile are well known. Tiles are also made of glass and are used in the place of skylights.

Tiles may be laid (1) on plank sheathing, (2) on reinforced concrete sheathing, or (3) may be supported directly on angle sub-purlins as shown in Fig. 13. Tiles are laid on sheathing in the same manner as slates.

The roof shown in Fig. 13 was constructed as follows: Terra-cotta tiles, manufactured by the Ludowici Roofing Tile Co., Chicago, Ill., were laid directly on the angle sub-purlins, every fourth tile being secured to the angle sub-purlins by a piece of copper wire. The tiles were interlocking, requiring no cement except in exceptional cases. The tiles were 9 × 16 in. in size; 135 being sufficient to lay a square of 100 sq. ft. of roof. These tiles weigh from 750 to 800 lb. per square, and cost about $6.00 per square at the factory. Skylights in this roof were made by substituting glass tiles for the terra-cotta tiles. This and similar tile have been used in this manner on a large number of mills and train sheds with excellent results.

Tile roofs laid without sheathing do not ordinarily condense the steam on the inner surface of the roof unless the tiles are glazed, although several cases have been brought to the author's attention where the condensation has caused trouble with tile roofs made of porous tiles. Anti-condensation roof lining should be used where there is danger of excessive sweating, or a porous tile should be used that is known to be non-sweating.

TIN ROOFING.—Two sizes of tin plates are in common use, 14 in. × 20 in. and 20 in. × 28 in., the latter size being most used. Tin sheets are made in several thicknesses, the IC, or No. 29 gage weighing 8 ounces to the sq. ft., and the IX, or No. 27 gage weighing 10 ounces to the sq. ft., being the most used. The standard weight of a box of 112 sheets, 14 × 20 size is 108 lb. for IC plate, and 136 lb. for IX plate. Boxes containing imperfect sheets or "wasters" are marked ICW or IXW. Every sheet should be stamped with the name of the brand and the thickness. The value of tin roofing depends upon the amount of tin used in coating and the uniformity with which the iron has been coated. The amount of tin used varies from 8 to 47 lb. for a box of 20 × 28 size containing 112 sheets.

Tin roofing is laid (1) with a flat seam, or (2) with a standing seam. In the former method the sheets of tin are locked into each other at the edges, the seam is flattened and fastened with tin cleats or is nailed firmly and is soldered water tight. Rosin is the best flux for soldering, although some tinners recommend the use of diluted chloride of zinc. For flat roofs the tin should be locked and soldered at all joints, and should be secured by tin cleats and not by nails. For steep roofs the tin is commonly put on with standing seams, not soldered, running with the pitch of the roof, and with cross-seams double locked and soldered. One or two layers of tar paper should be placed between the sheathing and the tin.

The under side of the sheets should be painted before laying. Tin roofs should be painted every two or three years. If kept well painted a tin roof should last 25 to 30 years.

For flat seam roofing, using ½ in. locks, a box of 14 × 20 tin will cover 192 sq. ft., and for standing seam, using ⅜ in. locks and turning 1¼ and 1½ in. edges, making 1 in. standing seams,

it will lay 168 sq. ft. For flat seam roofing, using ½ in. locks, a box of 20 × 28 tin will lay about 399 sq. ft., and for standing seam, using ⅜ in. locks and turning 1¼ and 1½ in. edges, making 1 in. standing seams, it will lay about 365 sq. ft.

TAR AND GRAVEL ROOF.—Tar and gravel roofs are called three-, four-, five-ply, etc., depending upon the number of layers of roofing felt. Tar and gravel roofs may be laid upon timber sheathing or upon concrete slabs. For details of a tar and gravel roof see Fig. 23. The following specifications are taken from the author's "Specifications for Steel Frame Buildings."

Specifications for Five-Ply Tar and Gravel Roof on Timber Sheathing.—The materials used in making the roof are 1 (one) thickness of sheathing paper or unsaturated felt, 5 (five) thicknesses of saturated felt weighing not less than 15 (fifteen) lb. per square of one hundred (100) sq. ft., single thickness, and not less than one hundred and twenty (120) lb. of pitch, and not less than four hundred (400) lb. of gravel or three hundred (300) lb. of slag from ¼ to ⅝ in. in size, free from dirt, per square of one hundred (100) sq. ft. of completed roof.

The material shall be applied as follows: First, lay the sheathing or unsaturated felt, lapping each sheet one in. over the preceding one. Second, lay two (2) thicknesses of tarred felt, lapping each sheet seventeen (17) in. over the preceding one, nailing as often as may be necessary to hold the sheets in place until the remaining felt is applied. Third, coat the entire surface of this two-ply layer with hot pitch, mopping on uniformly. Fourth, apply three (3) thicknesses of felt, lapping each sheet twenty-two (22) in. over the preceding one, mopping with hot pitch the full width of the 22 in. between the plies, so that in no case shall felt touch felt. Such nailing as is necessary shall be done so that all nails will be covered by not less than two plies of felt; fifth, spread over the entire surface of the roof a uniform coating of pitch, into which, while hot, imbed the gravel or slag. The gravel or slag in all cases must be dry.

Specifications for Five-Ply Tar and Gravel Roof on Concrete Sheathing.—The materials used shall be the same as for tar and gravel roof on timber sheathing. except that the one thickness of sheathing paper or unsaturated felt may be omitted.

The materials shall be applied as follows: First, coat the concrete with hot pitch, mopped on uniformly. Second, lay two (2) thicknesses of tarred felt, lapping each sheet seventeen (17) in. over the preceding one, and mop with hot pitch the full width of the 17-in. lap, so that in no case shall felt touch felt. Third, coat the entire surface with hot pitch, mopped on uniformly. Fourth, lay three (3) thicknesses of felt, lapping each sheet twenty-two (22) in. over the preceding one, mopping with hot pitch the full width of the 22-in. lap between the plies, so that in no case shall felt touch felt. Fifth, spread the entire surface of the roof with a uniform coat of pitch, into which, while hot, imbed gravel or slag.

Cost of Five-Ply Tar and Gravel Roofing.*—The cost of a round house roof in the middle west, based on 1912 prices and containing 500 squares of five-ply tar and gravel roofing, was as follows.

Cost per square of 100 sq. ft. not including fixed charges or profit, not including sheathing.

Sheathing paper, 5 lb.	$0.12
Pitch, 155 lb. at 60 cents per 100 lb.	0.93
Felt, 85 lb. at $1.65 per 100 lb.	1.40
Nails and caps	0.05
Cleats for flashing	0.05
Gravel (about one-seventh yard)	.23
Labor, including hauling, board and railroad fare	1.15
Total cost per square	$3.93

CEMENT ROOFING TILE.—Cement tile are made of Portland cement and clean, sharp sand and are reinforced with steel rods.

Data for "Bonanza" cement tile, manufactured by the American Cement Tile Mfg. Co., Pittsburgh, Pa., are given in Fig. 24a. The exposed surface of the tile is Indian red in color, while the underside has a cement finish. The least desirable slope of roof is a pitch of one-fifth. Data for Federal Cement tile, manufactured by the Federal Cement Tile Co., Chicago, Ill., are given in Fig. 24b, and in the upper part of Fig. 24c. Cement roofing tile have been very extensively used for industrial plants. The cement tile have the following advantages: (a) are fire resisting; (b) require very simple roof construction; (c) require no sheathing; (d) are non-

* Am. Ry. Eng. Assoc., Vol. 14, p. 852.

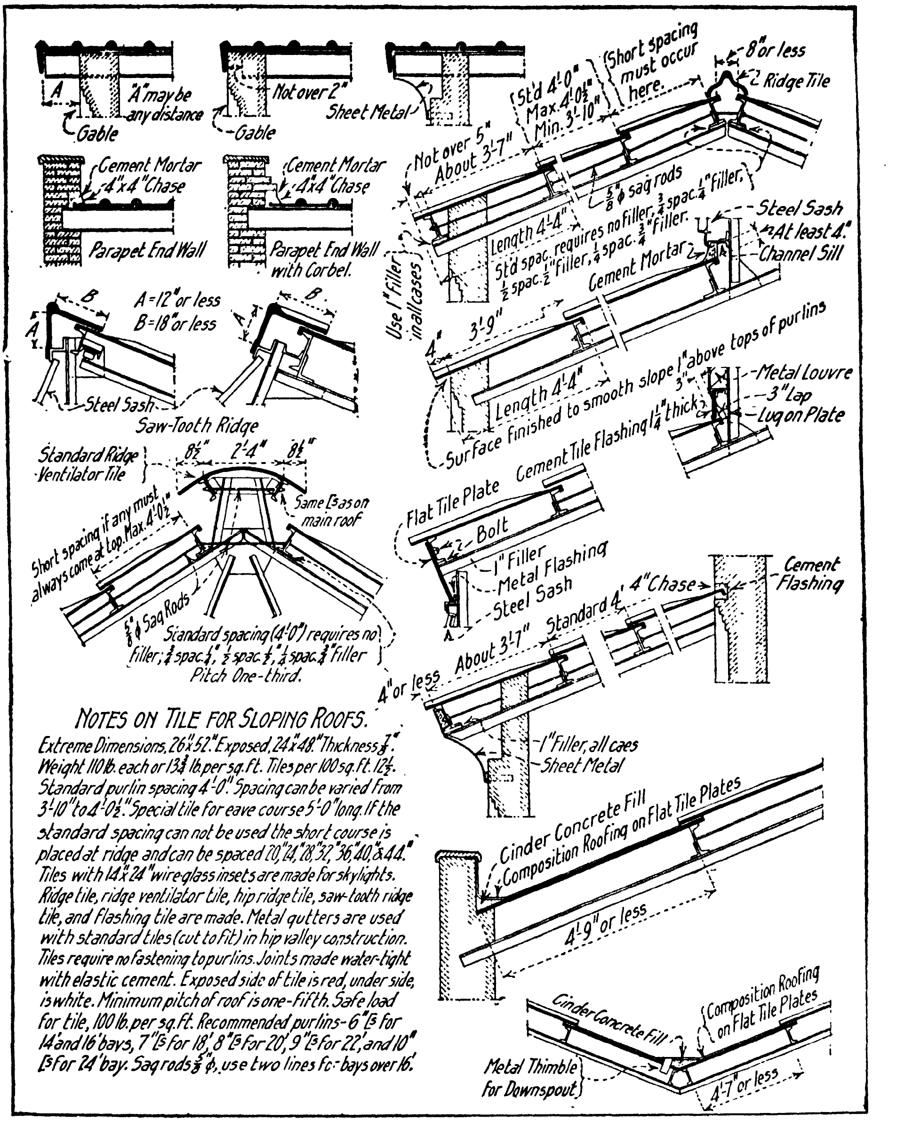


FIG. 24a. DATA FOR "BONANZA" CEMENT TILE.

conductors; (e) may be erected rapidly; (f) the first cost is low for a permanent type of roof, (g) maintenance is low.

Gypsum Roofing Tile.—Gypsum roofing tile made by the United States Gypsum Company, Chicago, are sold under the trade name of Pyrobar Gypsum Roof Tile. The tile are 12 in. wide and 30 in. long, and weigh 13 lb. per sq. ft. Data taken from the catalog for rafters and purlins for Pyrobar Gypsum Roof Tile are given in the lower part of Fig. 24c. Gypsum roof tile have recently been used on buildings for the Navy Department at Norfolk, Va. The following advan-

tages of gypsum roof slabs were given by L. M. Cox, U. S. N., Engineering News, Jan. 25, 1917: (a) Light weight; (b) rapid construction; (c) roof slab is non-conductor and non-condensing; (d) is fire resisting; (e) shows few cracks; (f) low cost of maintenance. Gypsum roofing tile are made by several firms, and are also made at the building site.

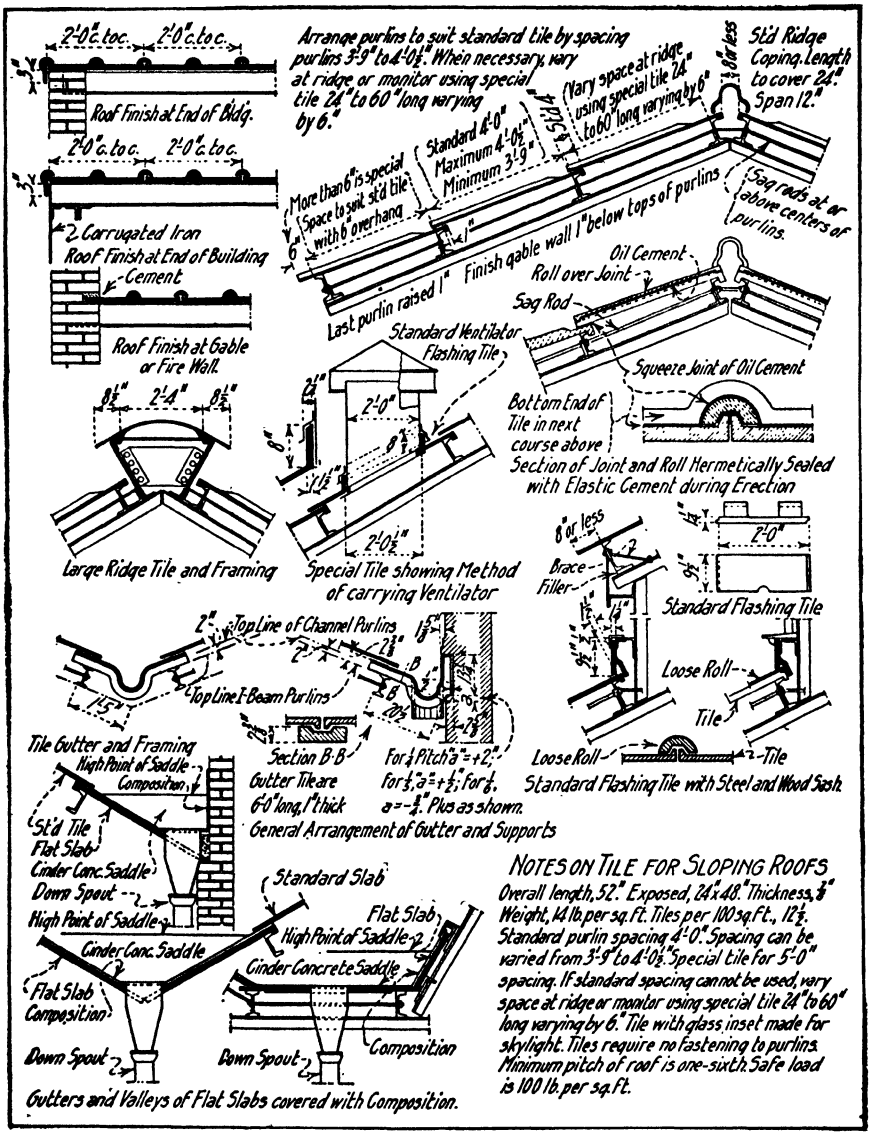
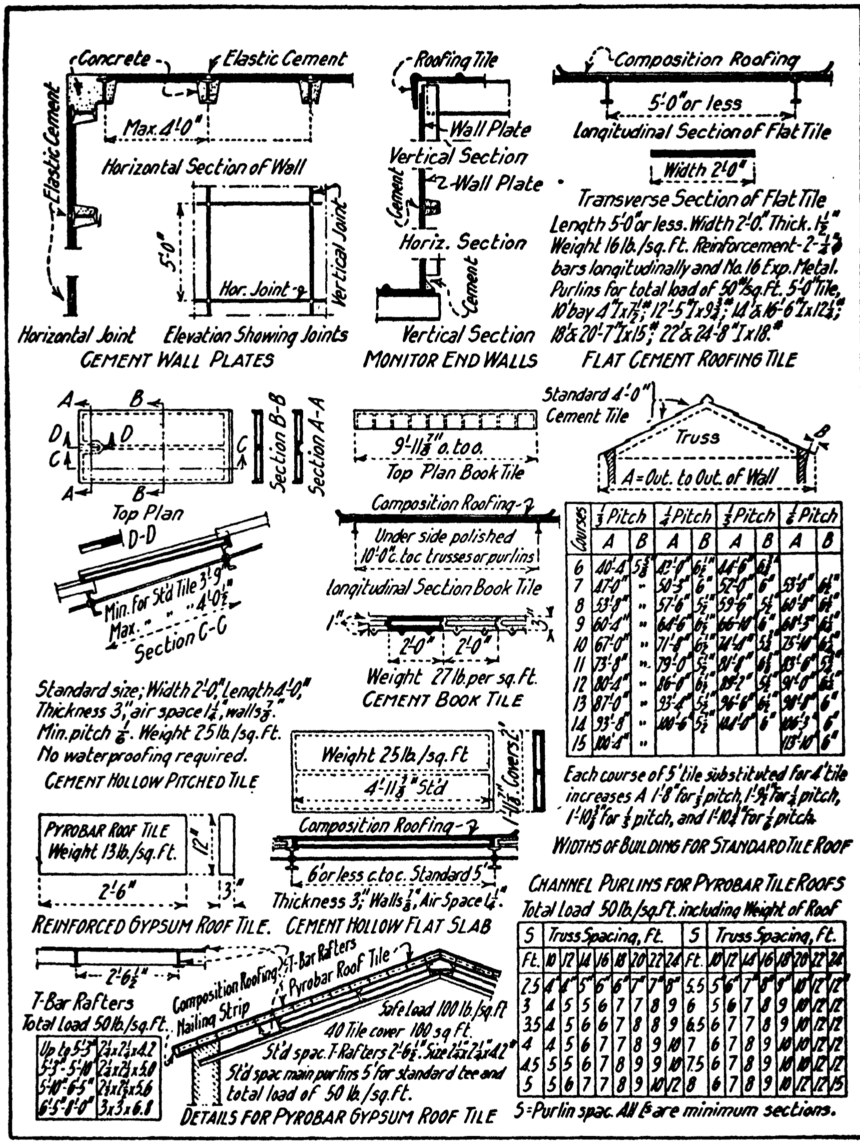


FIG. 24b. DATA FOR FEDERAL CEMENT TILES.

Sag Rods.—The purlins in roof framing carrying corrugated steel roofing should be supported by one sag rod for spans of 20 ft. and under, and by two sag rods for spans of over 20 ft. The purlins in roof framing carrying tile roofing should be supported by one sag rod for spans of



Up to 5'-3"	2¼x2¼x4.2
5'-3"-5'-10"	2¼x2¼x5.0
5'-10"-6'-5"	2½x2½x5.6
6'-5"-8'-0"	3x3x6.8

Courses	⅓ Pitch A	⅓ Pitch B	¼ Pitch A	¼ Pitch B	⅕ Pitch A	⅕ Pitch B	⅙ Pitch A	⅙ Pitch B
6	40'-4"	5⅝"	43'-0"	6½"	44'-6"	6¼"		
7	47'-0"	"	50'-3"	6"	52'-0"	6"	53'-0"	6½"
8	53'-8"	"	57'-6"	5½"	59'-6"	5½"	60'-8"	6½"
9	60'-4"	"	64'-6"	6½"	66'-10"	6"	68'-5"	6½"
10	67'-0"	"	71'-8"	6½"	74'-4"	5¾"	75'-10"	6½"
11	73'-8"	"	79'-0"	5½"	81'-8"	6¼"	83'-6"	5½"
12	80'-4"	"	86'-0"	6½"	89'-2"	5½"	91'-0"	6½"
13	87'-0"	"	93'-4"	5½"	96'-6"	6½"	98'-8"	6"
14	93'-8"	"	100'-6"	5½"	104'-0"	6"	106'-3"	6"
15	100'-4"	"					113'-10"	6"

S Ft.	10	12	14	16	18	20	22	24	S Ft.	10	12	14	16	18	20	22	24
2.5	4"	4"	5"	6"	6"	7"	7"	8"	5.5	5"	6"	7"	8"	9"	10	12	12"
3	4	5	5	6	7	7	8	9	6	5	6	7	8	9	10	12	12
3.5	4	5	6	6	7	8	8	9	6.5	6	7	7	8	9	10	12	12
4	4	5	6	7	7	8	9	10	7	6	7	8	9	10	10	12	12
4.5	5	5	6	7	8	9	9	10	7.5	6	7	8	9	10	10	12	12
5	5	6	7	7	8	9	10	12	8	6	7	8	9	10	12	12	15

FIG. 24C. DATA FOR FEDERAL CEMENT TILE (UPPER PART), AND DATA FOR PYROBAR GYPSUM TILE (LOWER PART).

14 ft. or under, and by two sag rods placed at the third points of the span for spans of more than 14 ft. Details of roof framing are given in Fig. 24d.

The specifications for sag rods in specifications for steel frame buildings in the latter part of this chapter are as follows:—

Sag Rods.—With a steel corrugated roof one sag rod, at the center, shall be used for purlin spans of 20 ft. or less, and two sag rods, spaced at the third points, for purlin spans of more than 20 ft. With clay tile, cement tile, slate, gypsum or similar roofs, one sag rod shall be used for purlin spans of 14 ft. or less, and two sag rods spaced at the third points for spans of more than 14 ft. Where one sag rod is used, the sag rod on each side of the roof in any panel shall be rigidly connected through the ridge purlins. Where two sag rods are used in any panel, each sag rod shall be rigidly connected with the peak of the nearest truss by means of a diagonal sag rod in the upper purlin space. Sag rods need not be used in roofs with a pitch of 3 in. in 12 in., or less. With corrugated steel siding, one sag rod shall be used for all girt spacings of 20 ft. or less, and two sag rods, spaced at third points, for girt spacings of more than 20 ft.

Sag rods shall be designed to carry the component of the dead load of the purlins and roof covering and the maximum snow load parallel to the roof surface, with a unit stress of 16,000 lb. per sq. in. on net section. Sag rods for the sides shall be designed to carry the weight of the side framing and covering with the same allowable unit stresses as for sag rods for purlins. If sag rods are not upset, the net section shall be taken as the section having a diameter 1/16 in. less than the diameter of the root of the thread. The minimum size of sag rods shall have a diameter of ½ in. if the ends are upset, or ⅝ in. if the ends are not upset.

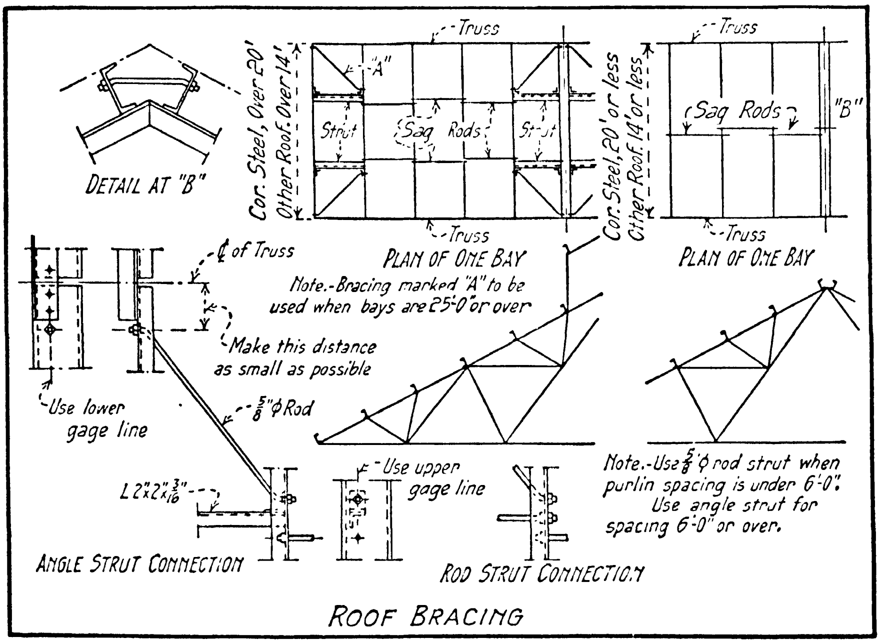


FIG. 24d. DETAILS OF ROOF FRAMING.

SHOP FLOORS.—Floors for industrial plants may be placed on a foundation resting directly on the ground or may be self supporting. Several examples of shop floors that rest on the ground are shown in Fig. 25. Standard specifications for a cement floor and for a wood floor on a tar concrete base follow.

The following specifications are from the author's "Specifications for Steel Frame Buildings."

Specifications for Cement Floor on a Concrete Base. Materials.—The cement used shall be first-class Portland cement, and shall pass the standards of the American Society for Testing Materials. The sand for the top finish shall be clean and sharp and shall be retained on a No. 30 sieve and shall have passed the No. 20 sieve. Broken stone for the top finish shall pass a ½ in.

screen and shall be retained on the No. 20 screen. Dust shall be excluded. The sand for the base shall be clean and sharp. The aggregate for the base shall be of broken stone or gravel and shall pass a 2 in. ring.

Base.—On a thoroughly tamped and compacted subgrade the concrete for the base shall be laid and thoroughly tamped. The base shall not be less than 2½ in. thick. Concrete for the base shall be thoroughly mixed with sufficient water so that some tamping is required to bring the moisture to the surface. If old concrete is used for the base the surface shall be roughened

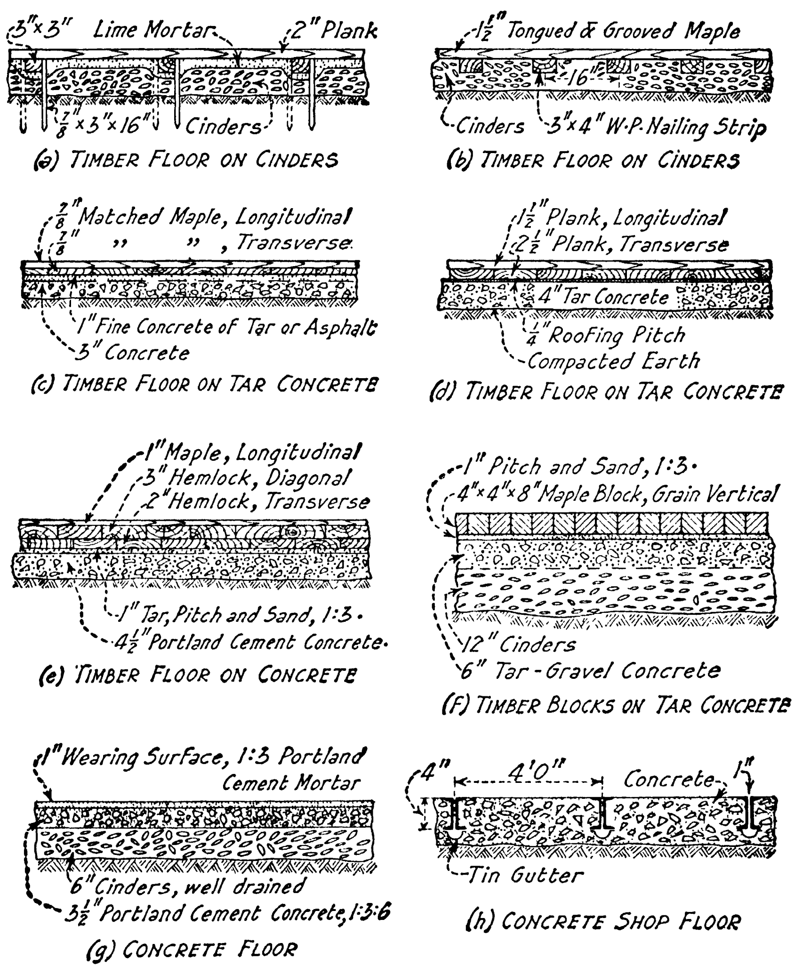


FIG. 25. EXAMPLES OF GROUND SHOP FLOORS.

and thoroughly cleaned so that the new mortar will adhere. The roughened surface of old concrete shall then be thoroughly wet so that the base will not draw water from the finish when the latter is applied. Before scrubbing the base with grout the excess water shall be removed.

Finish.—With old concrete the surface of the base shall first be scrubbed with a thin grout of pure cement, rubbed in with a broom. On top of this, before the thin coat is set, a coat of finish mixed in the proportions of one part Portland cement, one part stone broken to pass a ¼ in. ring, and one part sand shall be troweled on using as much pressure as possible, so that it will take a firm bond. After the finish has been applied to the desired thickness it should be screeded and floated to a true surface. Between the time of initial and final set it shall be finished by

skilled workmen with steel trowels and shall be worked to a final surface. Under no condition shall a dryer be used, nor shall water be added to make the material work easily.

Specifications for Wood Floor on a Tar Concrete Base. Floor Sleepers.—Sleepers for carrying the timber floor shall be 3 in. × 3 in. placed 18 in. c. to c. After the subgrade has been thoroughly tamped and rolled to an elevation of 4½ in. below the tops of the sleepers, the sleepers shall be placed in position and supported on stakes driven in the subgrade. Before depositing the tar concrete the sleepers must be brought to a true level.

Tar Concrete Base.—The tar concrete base shall be not less than 4½ in. thick and shall be laid as follows: First, a layer three (3) in. thick of coarse, screened gravel thoroughly mixed with tar, and tamped to a hard level surface. Second, on this bed spread a top dressing 1½ in. thick of sand heated and thoroughly mixed with coal tar pitch, in the proportions of one (1) part pitch to three (3) parts tar. The gravel, sand and tar shall be heated to from 200 to 300 degrees F., and shall be thoroughly mixed and carefully tamped into place.

Plank Sub-Floor.—The floor plank shall be of sound hemlock or pine not less than 2 in. thick, planed on one side and one edge to an even thickness and width. The floor plank is to be toe-nailed with 4 in. wire nails.

Finished Flooring.—The finished flooring is to be of maple of clear stock, ⅞ in. finished thickness, thoroughly air and kiln dried and not over 4 in. wide. The flooring is to be planed to an even thickness, the edges jointed, and the underside channeled or ploughed. The finished floor is to be laid at right angles to the sub-floor, and each board neatly fitted at the ends, breaking joints at random. The floor is to be final nailed with 10 d. or 3 in. wire nails, nailed in diagonal rows 16 in. apart across the boards, with two (2) nails in each row in every board. The floor to be finished off perfectly smooth on completion.

The finished flooring is not to be taken into the building or laid until the tar concrete base and sub-plank floor are thoroughly dried.

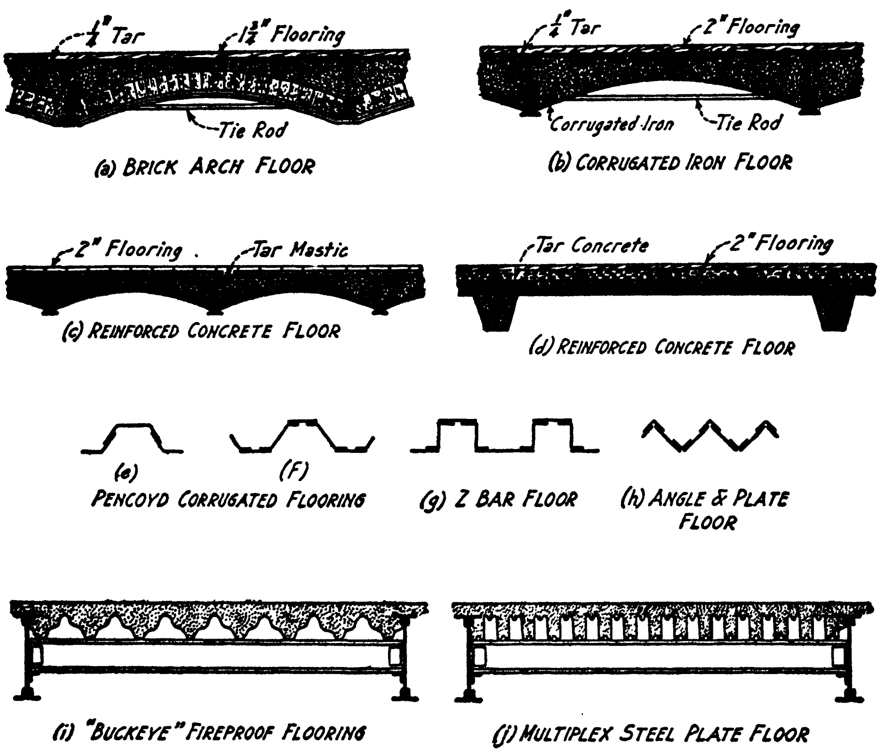


Fig. 26. Examples of Shop Floors Above Ground.

Shop floors above ground may be made of timber resting on beams, of brick arch construction, (a) Fig. 26, of concrete with corrugated steel arch centers as shown in (b), of reinforced con-

crete as shown in (*c*) and (*d*), of steel filled with concrete as shown in (*e*), (*f*), (*g*), (*h*), or of concrete reinforced with Buckeye flooring as shown in (*i*) or Multiplex flooring as shown in (*j*).

Timber Floors.—The Yellow Pine Manufacturers Association has calculated the safe span of yellow pine when used for mill floors with fiber stresses of 1,200 to 1,800 lb. per sq. in. for live loads of 100 to 300 lb. per sq. ft. in addition to the weight of the floor, Table V. In the line marked "Deflection" is given the span which has a maximum deflection of one thirtieth of an inch per foot of span for the various live loads. The modulus of elasticity of timber was taken as 1,684,800 lb. per sq. in. The table may be used for any kind of timber by using the proper working stress. The maximum spans for fiber stresses less than 1,200 lb. per sq. in. may be found as follows: Required the maximum safe span for a timber floor 2⅝ in. thick for a fiber stress of 800 lb. per sq. in. and a live load of 150 lb. per sq. ft. The span is approximately the same as for a fiber stress of 1,200 lb. per sq. in. and a live load of 225 lb. per sq. ft., = 6 ft. 11 in.; or for a fiber stress of 1,600 lb. per sq. in. and a live load of 300 lb. per sq. ft., = 6 ft. 11 in.

TABLE V.

Allowable Span for Timber Floors.

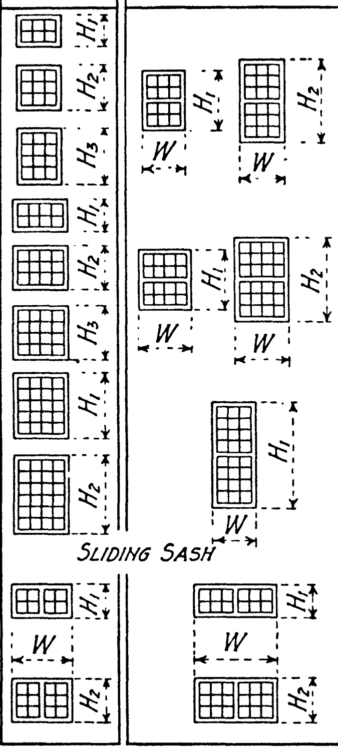
Yellow Pine Manufacturers Association.

Thickness in Inches.	Stress per Square Inch. Pounds.	SPAN IN FEET. Live Load in Pounds Per Square Foot.								
		100	125	150	175	200	225	250	275	300
1⅝	1,200	6′ 4″	5′ 8″	5′ 3″	4′ 10″	4′ 6″	4′ 4″	4′ 1″	3′ 11″	3′ 9″
	1,300	6′ 7″	5′ 11″	5′ 5″	5′ 0″	4′ 9″	4′ 6″	4′ 3″	4′ 1″	3′ 10″
	1,500	7′ 1″	6′ 4″	5′ 10″	5′ 5″	5′ 1″	4′ 10″	4′ 7″	4′ 4″	4′ 2″
	1,600	7′ 4″	6′ 7″	6′ 0″	5′ 7″	5′ 3″	5′ 0″	4′ 8″	4′ 6″	4′ 4″
	1,800	7′ 9″	7′ 0″	6′ 5″	5′ 11″	5′ 7″	5′ 3″	5′ 0″	4′ 9″	4′ 7″
	Deflection	4′ 8″	4′ 4″	4′ 1″	3′ 11″	3′ 9″	3′ 7″	3′ 5½″	3′ 4½″	3′ 3″
2⅝	1,200	10′ 1″	9′ 1″	8′ 4″	7′ 9″	7′ 3″	6′ 11″	6′ 6″	6′ 3″	6′ 0″
	1,300	10′ 6″	9′ 6″	8′ 8″	8′ 1″	7′ 7″	7′ 2″	6′ 10″	6′ 6″	6′ 3″
	1,500	11′ 3″	10′ 2″	9′ 4″	8′ 8″	8′ 2″	7′ 8″	7′ 4″	7′ 0″	6′ 8″
	1,600	11′ 8″	10′ 6″	9′ 8″	8′ 11″	8′ 5″	7′ 11″	7′ 7″	7′ 2″	6′ 11″
	1,800	12′ 4″	11′ 2″	10′ 3″	9′ 6″	8′ 11″	8′ 5″	8′ 0″	7′ 8″	7′ 4″
	Deflection	7′ 5½″	6′ 11½″	6′ 7″	6′ 3″	6′ 0″	5′ 9½″	5′ 7″	5′ 5″	5′ 3″
3⅝	1,200	......	......	11′ 3″	10′ 7″	10′ 0″	9′ 5″	9′ 0″	8′ 7″	8′ 3″
	1,300	......	......	11′ 8″	11′ 0″	10′ 5″	9′ 10″	9′ 4″	8′ 11″	8′ 7″
	1,500	......	......	12′ 7″	11′ 10″	11′ 2″	10′ 7″	10′ 0″	9′ 7″	9′ 2″
	1,600	......	......	13′ 0″	12′ 3″	11′ 6″	10′ 11″	10′ 4″	9′ 11″	9′ 6″
	1,800	......	......	13′ 9″	13′ 0″	12′ 3″	11′ 7″	11′ 0″	10′ 6″	10′ 1″
	Deflection	10′ 2½″	9′ 6½″	9′ 0″	8′ 7″	8′ 3″	7′ 11½″	7′ 8″	7′ 5½″	7′ 3″
4⅝	1,200	......	......	......	......	12′ 7″	11′ 11″	11′ 4″	10′ 10″	10′ 5″
	1,300	......	......	......	......	13′ 2″	12′ 5″	11′ 10″	11′ 4″	10′ 10″
	1,500	......	......	......	......	14′ 1″	13′ 4″	12′ 9″	12′ 2″	11′ 8″
	1,600	......	......	......	......	14′ 7″	13′ 9″	13′ 2″	12′ 7″	12′ 1″
	1,800	......	......	......	......	15′ 5″	14′ 8″	14′ 2″	13′ 4″	12′ 9″
	Deflection	12′ 11″	12′ 1″	11′ 5¼″	10′ 11″	10′ 6″	10′ 1″	9′ 9″	9′ 6″	9′ 2½″
5⅝	1,200	......	......	......	......	15′ 3″	14′ 5″	13′ 9″	13′ 2″	12′ 7″
	1,300	......	......	......	......	15′ 10″	15′ 0″	14′ 4″	13′ 8″	13′ 1″
	1,500	......	......	......	......	17′ 1″	16′ 1″	15′ 4″	14′ 8″	14′ 1″
	1,600	......	......	......	......	17′ 7″	16′ 8″	15′ 10″	15′ 2″	14′ 7″
	1,800	......	......	......	......	18′ 8″	17′ 8″	16′ 10″	16′ 1″	15′ 5″
	Deflection	13′ 7″	12′ 8½″	12′ 0½″	11′ 6″	11′ 0½″	10′ 8″	10′ 4″	10′ 9″	10′ 9″

Waterproofing.—For methods of waterproofing floors, walls, etc., see methods of waterproofing bridge floors in Chapter IV.

DIMENSIONS FOR GLAZED WOOD SASH

Size of Glass	Width W	Height H_1	Height H_2	Height H_3	Single Sash	Double Hung Sash	Height H_2	Height H_1	Width W	Size of Glass
10"×12"	2'11¼"	2'5⅝"	3'6"	4'6⅜"			6'8¼"	4'7½"	2'11¼	10"×12"
12×12	3-5¼	2-5⅝	3-6	4-6⅜			6-8¼	4-7½	3-5¼	12×12
10×14	2-11¼	2-9⅝	4-0	5-2⅜			7-8¼	5-3½	2-11¼	10×14
12×14	3-5¼	2-9⅝	4-0	5-2⅜			7-8¼	5-3½	3-5¼	12×14
10×16	2-11¼	3-1⅝	4-6	5-10⅜			8-8¼	5-11½	2-11¼	10×16
12×16	3-5¼	3-1⅝	4-6	5-10⅜			8-8¼	5-11½	3-5¼	12×16
14×16	3-11¼	3-1⅝	4-6	5-10⅜			8-8¼	5-11½	3-11¼	14×16
10×12	3-9⅝	2-5⅝	3-6	4-6⅜			6-8¼	4-7½	3-9⅝	10×12
12×12	4-5⅝	2-5⅝	3-6	4-6⅜			6-8¼	4-7½	4-5⅝	12×12
10×14	3-9⅝	2-9⅝	4-0	5-2⅜			7-8¼	5-3½	3-9⅝	10×14
12×14	4-5⅝	2-9⅝	4-0	5-2⅜			7-8¼	5-3½	4-5⅝	12×14
10×16	3-9⅝	3-1⅝	4-6	5-10⅜			8-8¼	5-11½	3-9⅝	10×16
12×16	4-5⅝	3-1⅝	4-6	5-10⅜			8-8¼	5-11½	4-5⅝	12×16
14×16	5-1⅝	3-1⅝	4-6	5-10⅜			8-8¼	5-11½	5-1⅝	14×16
10×12	3-9⅝	5-6¾	6-8¼					8-9	2-11¼	10×12
12×12	4-5⅝	5-6¾	6-8¼					8-9	3-5¼	12×12
10×14	3-9⅝	6-4¾	7-8¼					10-1	2-11¼	10×14
12×14	4-5⅝	6-4¾	7-8¼					10-1	3-5¼	12×14
10×16	3-9⅝	7-2¾	8-8¼					11-5	2-11¼	10×16
12×16	4-5⅝	7-2¾	8-8¼					11-5	3-5¼	12×16
14×16	5-1⅝	7-2¾	8-8¼					11-5	3-11¼	14×16
					SLIDING SASH					
10×12	3-11½	2-5⅝	3-6				3-6	2-5⅝	5-8¼	10×12
12×12	4-7½	2-5⅝	3-6				3-6	2-5⅝	6-8¼	12×12
10×14	3-11½	2-9⅝	4-0				4-0	2-9⅝	5-8¼	10×14
12×14	4-7½	2-9⅝	4-0				4-0	2-9⅝	6-8¼	12×14
10×16	3-11½	3-1⅝	4-6				4-6	3-1⅝	5-8¼	10×16
12×16	4-7½	3-1⅝	4-6				4-6	3-1⅝	6-8¼	12×16
14×16	5-3½	3-1⅝	4-6				4-6	3-1⅝	7-8¼	14×16



QUALITY OF GLASS

"B" American Single Strength				"B" American Double Strength		
10"×12"	12"×12"	10"×14"	12"×14"	10"×16"	12"×16"	14"×16"

All sash to be 1⅜" thick, except Sliding Sash, Pivoted Sash, and Single Sash (or one half of Double Sash) exceeding 4'6" high or 4'0" wide, which should be made 1¾" thick. Top Rails 2¼". Stiles 2¼". Bottom Rail 3". Muntins ⅜".

Pivoted Sash, 4 lights high or over, to have one Horizontal Muntin 1½" thick; all other Sash, 6 lights high or over, to have one Horizontal Muntin 1½" thick.

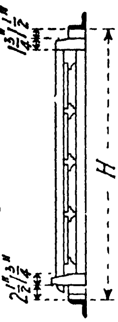
Pivoted Sash, 4 lights wide or over, to have one Vertical Muntin 1½" thick; all other Sash, 6 lights wide or over, to have one Vertical Muntin 1½" thick.

For Pivoted Sash 4 and 5 lights high or wide, add 1⅛" to figures given in above tables.

FIG. 27. DIMENSIONS AND DATA FOR GLAZED WOOD SASH. AMERICAN BRIDGE COMPANY.

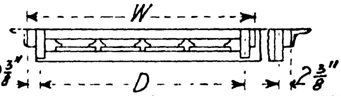
Height of Glass	No. of Lights High	Spacing H
12″	2	3′1⅝″
12	3	4-2
12	4	5-2⅜
12	5	6-2¾
12	6	7-4¼
12	7	8-4⅝
14	2	3-5⅝
14	3	4-8
14	4	5-10⅜
14	5	7-0¾
14	6	8-4¼
14	7	9-6⅝

Distance H = Girt Spacing for Fixed, Pivoted or Sliding Windows.



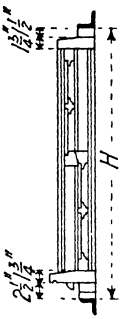
Width of Glass	No. of Lights Wide	Spacing W	Spacing D.	Width of Glass	No. of Lights Wide	Spacing W	Spacing D
10″	2	2′7⅞	2′2⅞″	12	2	2′11⅞″	2′6⅞″
10	3	3-6¼	3-1¼	12	3	4-0¼	3-7¼
10	4	4-4⅝	3-11⅝	12	4	5-0⅝	4-7⅝
10	5	5-3	4-10	12	5	6-1	5-8
10	6	6-2½	5-9½	12	6	7-2½	6-9½

W = Width of Single Pivoted, Fixed or Counter-balanced Window. Width of Continuous Window = No. of Windows × D, + 2⅜″ + 2⅜″ + (¼″ Clearance).



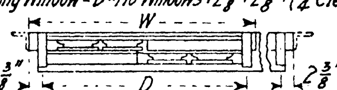
Height of Glass	No. of Lights High	Spacing H
12″	4	5′3¼″
12	6	7-4
12	8	9-4¾
12	10	11-5½
12	12	13-8½
14	4	5-11¼
14	6	8-4
14	8	10-8¾
14	10	13-1½
14	12	15-8½

Distance H = Girt Spacing for Counterbalanced Windows.



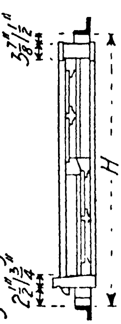
Width of Glass	No. of Lights Wide	Spacing W	Spacing D	Width of Glass	No. of Lights Wide	Spacing W	Spacing D
10″	4	4′6¼″	4′1¼″	12″	4	5′2¼″	4′9¼″
10	6	6-3	5-10	12	6	7-3	6-10
10	8	7-11¾	7-6¾	12	8	9-3¾	8-10¾
10	10	9-8½	9-3½	12	10	11-4½	10-11½
10	12	11-7½	11-2½	12	12	13-7½	13-2½

W = Width of Single Sliding Window. Width of Continuous Sliding Window = D × No Windows + 2⅜″ + 2⅜″ + (¼″ Clearance)



Height of Glass	No. of Lights High	Spacing H
12″	4	5′5⅜″
12	6	7-6⅛
12	8	9-6⅞
12	10	11-7⅝
12	12	13-10⅝
14	4	6-1⅜
14	6	8-6⅛
14	8	10-10⅞
14	10	13-3⅝
14	12	15-10⅝

Distance H = Girt Spacing for Double Hung Weighted Windows.



Width of Glass	No. of Lights Wide	Spacing W
10″	2	3′1⅜″
10	3	3-11¾
10	4	4-10⅛
10	5	5-8½
10	6	6-8

Width of Glass	No. of Lights Wide	Spacing W
12″	2	3′5⅝″
12	3	4-5¾
12	4	5-6⅛
12	5	6-6½
12	6	7-8

W = Width of Single Double Hung Weighted Window.

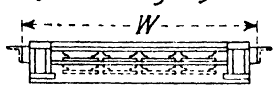


FIG. 28. DIMENSIONS FOR GLAZED WOOD SASH.
AMERICAN BRIDGE COMPANY.

WINDOWS AND SKY LIGHTS.—Mill and mine buildings should have an ample amount of glazing in the form of windows and sky lights. Plane glass is made in two thicknesses, single strength approximately $\frac{1}{16}$ in. thick, and double strength approximatley $\frac{1}{8}$ in. thick. Plane

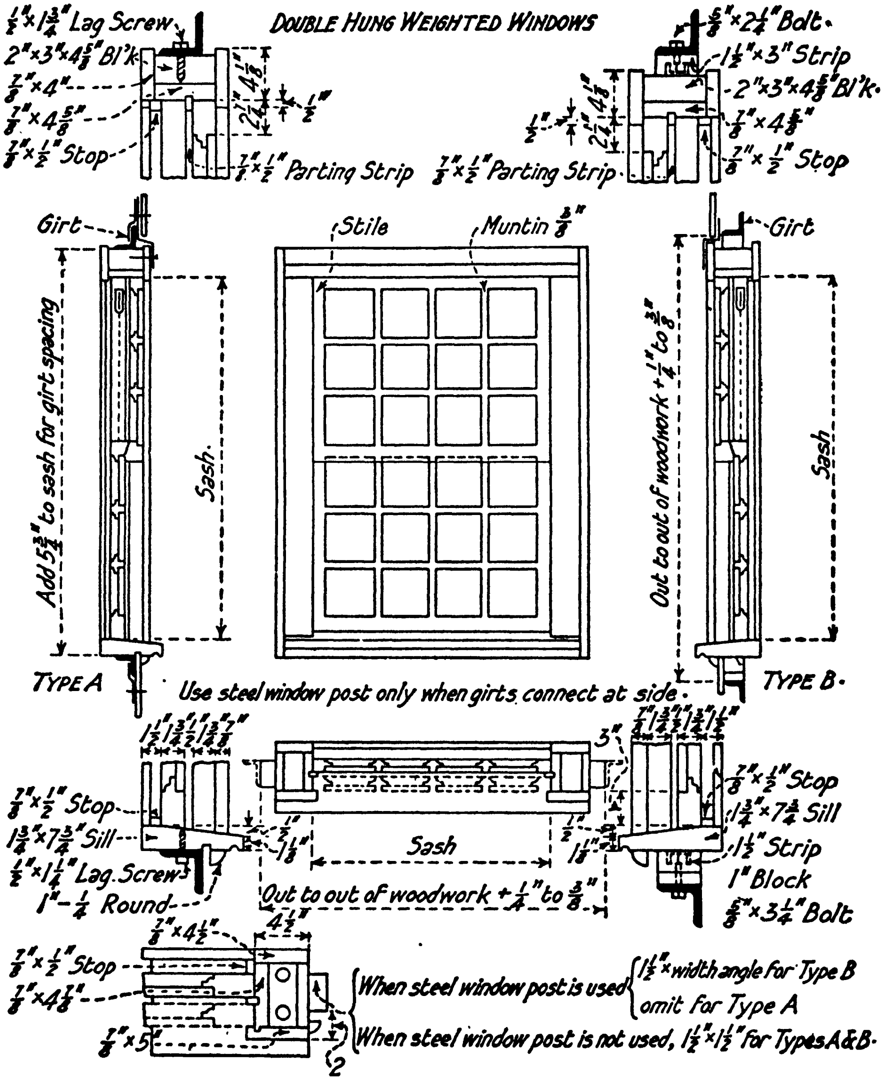
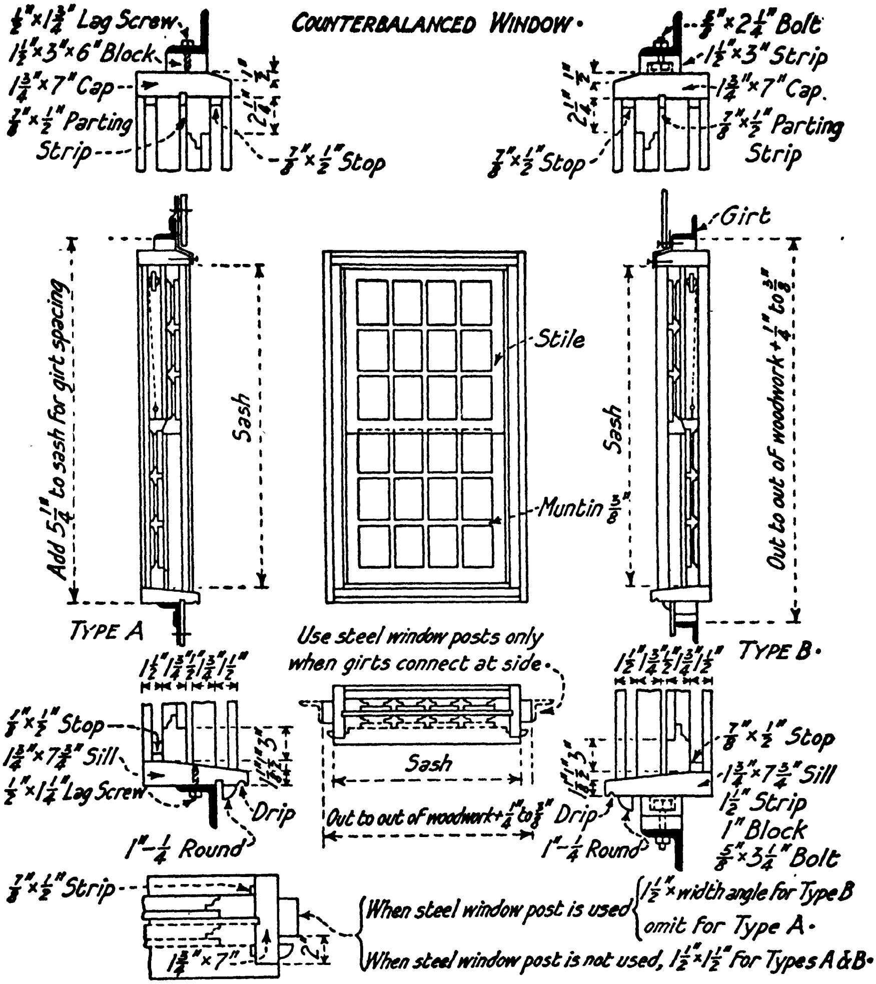


FIG. 29. DATA FOR DOUBLE HUNG WEIGHTED WINDOWS.
AMERICAN BRIDGE COMPANY.

glass is graded as AA, A, and B. The AA grade being the best and the B grade the poorest. Wire glass is $\frac{1}{16}$ in. or $\frac{1}{4}$ in. thick and may be obtained with a smooth surface, with factory ribs or prisms. For ordinary windows double strength glass gives very satisfactory results. For sky lights and where windows are liable to be broken, wire glass should be used. The best

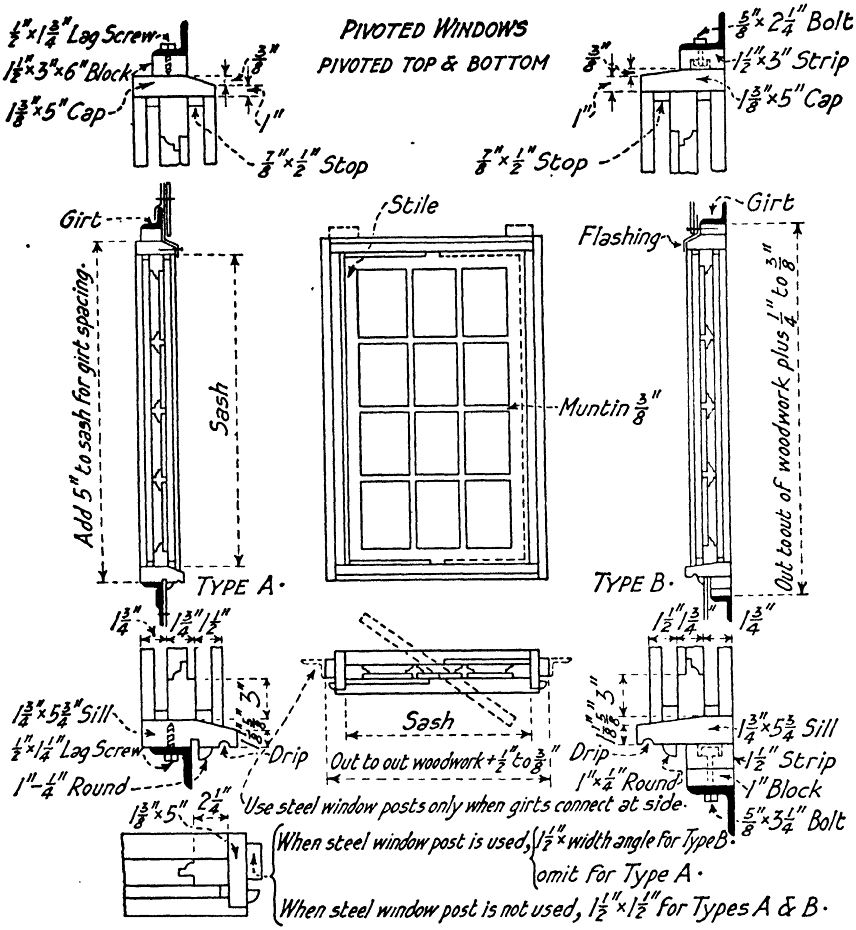


DIMENSIONS FOR WOOD FRAMES FOR TRIPLE HUNG COUNTERBALANCED WINDOW.

Height of Glass	No. Lights High	Spacing H	Height of Glass	No. Lights High	Spacing H
12"	6	7'5 1/8"	14"	6	8'5 1/8"
12	9	10-6 1/4	14	9	12-0 1/4
12	12	13-7 3/8	14	12	15-7 3/8
12	15	16-8 1/2	14	15	19-2 1/2
12	18	20-1	14	18	23-1

Distance H in table is Girt Spacing for Triple Hung Counterbalanced Windows. For width see sheet giving width of ordinary Counterbalanced Windows.

FIG 30. DATA FOR COUNTERBALANCED WINDOWS.
AMERICAN BRIDGE COMPANY.



DATA FOR SPACING BETWEEN STEEL WINDOW POSTS.
For Fixed, Pivoted and Counterbalanced Windows.

Glass	10" or 12"
Muntins (each)	3/8"
Stiles (each)	2 1/4"
Sash Clearance	1/4"
Jambs (each)	1 3/4"
Nailing Pieces (each)	1 1/2"
Frame Clearance	1/4"

For Sliding Windows use above data except no Sash Clearance, and add 2 1/4" for meeting rail.

FIG. 31. DATA FOR PIVOTED WINDOWS. AMERICAN BRIDGE COMPANY.

glass for glazing windows in industrial plants is "factory ribbed glass" with twenty-one ribs to the inch, the ribs being placed on the inside of the window. This glass is considerably more expensive than plane glass but is much more satisfactory.

Translucent fabric made by imbedding wire cloth in a translucent material made of linseed oil, is also used for glazing in industrial buildings. Translucent fabric will be charred by a live coal but is practically fire-proof. It shuts off part of the light, making it possible for men to work under it without shading.

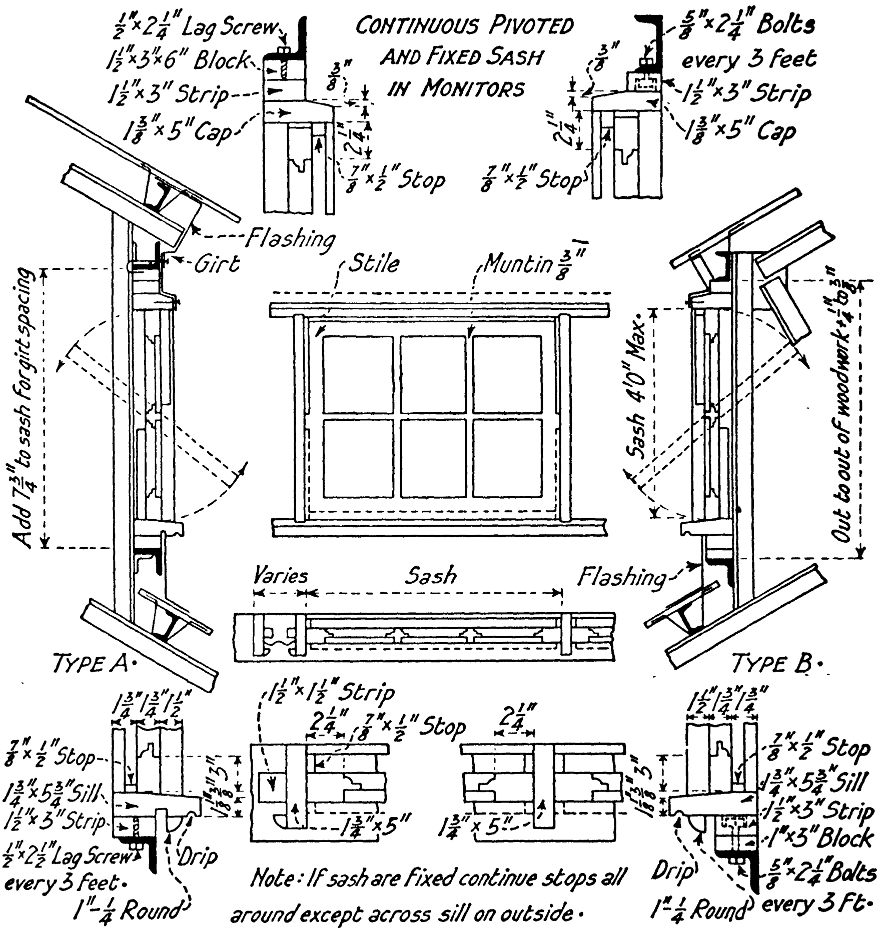
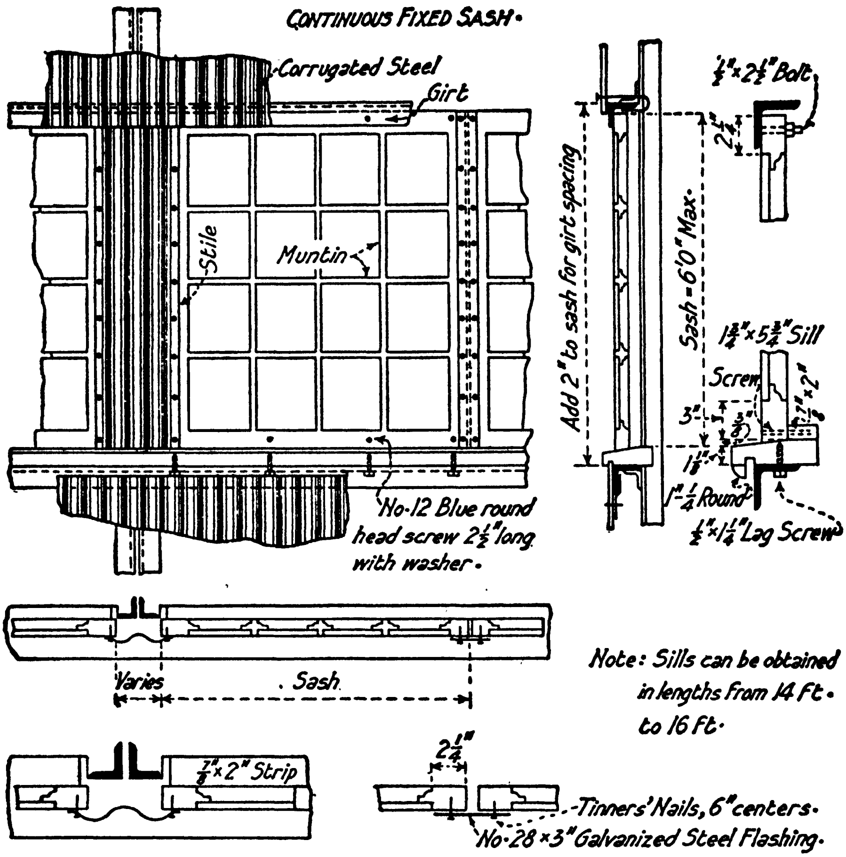


FIG. 32. DATA FOR CONTINUOUS PIVOTED AND FIXED SASH IN MONITORS. AMERICAN BRIDGE COMPANY.

The amount of glazed surface required in mill buildings depends upon the use to which the building is put, the material used in glazing, the location and the angle of the windows and sky lights, and the clearness of the atmosphere. It is common to specify that not less than 10 per cent of the exterior surface of mill buildings and 25 per cent of the exterior surface of machine shops should be glazed. Many industrial plants have as much as 60 per cent of the exterior walls of glass.



DATA FOR SPACING BETWEEN GIRTS
For Fixed, Pivoted and Sliding Windows.

Glass	12" or 14"	Sill and Head (each)	$1\frac{3}{4}$"
Sash Top Rail	$2\frac{1}{4}$"	Top Nailing Piece	$1\frac{1}{2}$"
Sash Bottom Rail	3"	Bottom Nailing Piece	$1\frac{1}{2}$"
Muntins (each)	$\frac{5}{8}$"	Block	1"
Sash Clearance	$\frac{1}{4}$"	Frame Clearance	$\frac{1}{4}$"

For Counterbalanced use above data except no Sash Clearance, and add $1\frac{1}{2}$" for meeting rail.

FIG. 33. DATA FOR CONTINUOUS FIXED SASH.
AMERICAN BRIDGE COMPANY.

Specifications for Windows and Skylights.—The requirements for windows and skylights as given in the author's specifications in the latter part of this chapter are as follows:—

Windows and Skylights.—Where buildings are lighted by windows the clear window area shall not be less than 20 per cent of the floor area, nor less than 10 per cent of the area of the entire exterior surface in mill buildings, nor less than 20 per cent of the area of the entire exterior

surface in machine shops, factories and other buildings in which men are required to work at machines. Skylights shall be used where the required window area cannot be provided in the sides and ends of buildings.

Where buildings are lighted by windows having the sills not more than 4 ft. above the floor, the span of the building shall not exceed 2 times the height of the top of the windows where buildings are lighted by windows in one side, or 4 times the height of the top of the windows where buildings are lighted by windows in both sides. Where the span of the building is greater than is permitted by the preceding requirement, the necessary illumination shall be provided either by prism glass in side walls or by skylights. Skylights shall have such an area and shall be so arranged that light coming through the skylight making an angle of not more than 45° with the vertical shall cover the entire horizontal area at a distance of 6 feet above the floor; or the light

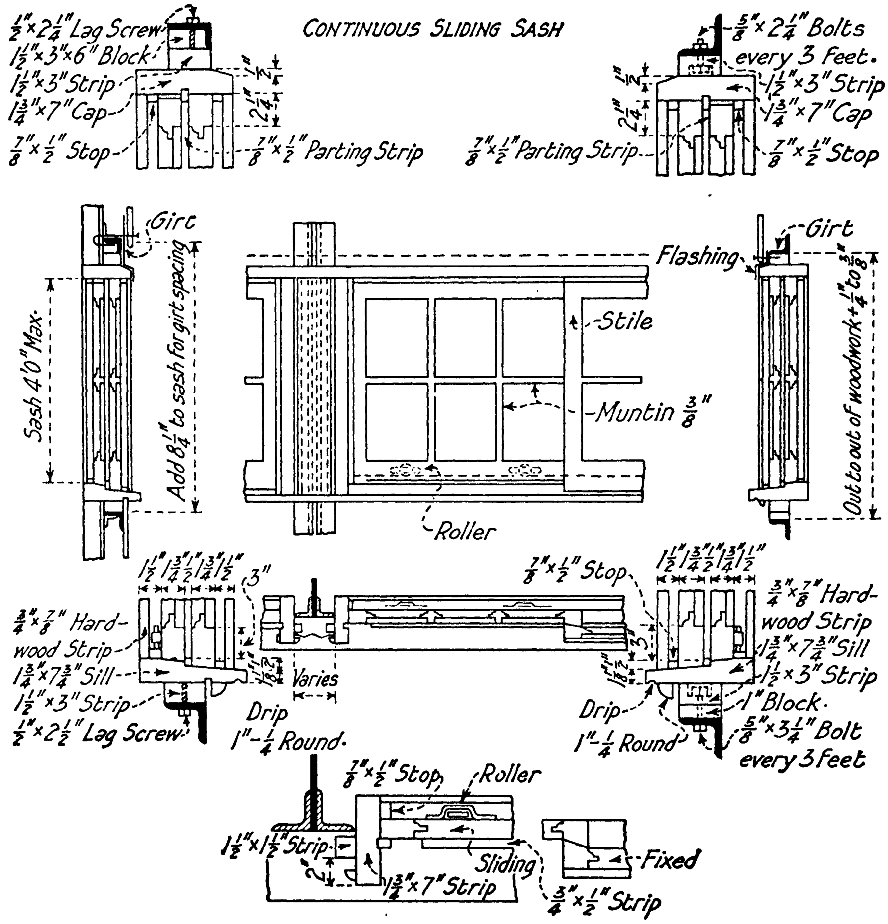


FIG. 34. DATA FOR CONTINUOUS SLIDING SASH.
AMERICAN BRIDGE COMPANY.

may be diffused by means of ribbed glass or prisms or by reflection from the ceiling to obtain equally satisfactory illumination. In saw tooth roofs the inner surface of the roof shall be light colored or shall be painted with a paint that will reflect the light and make the illumination uniform and effective. All windows or skylights admitting direct sunlight shall be provided with muslin or other satisfactory shades.

Details of glazed sash and window frames as adopted by the American Bridge Company are shown in Fig. 27 to Fig. 34.

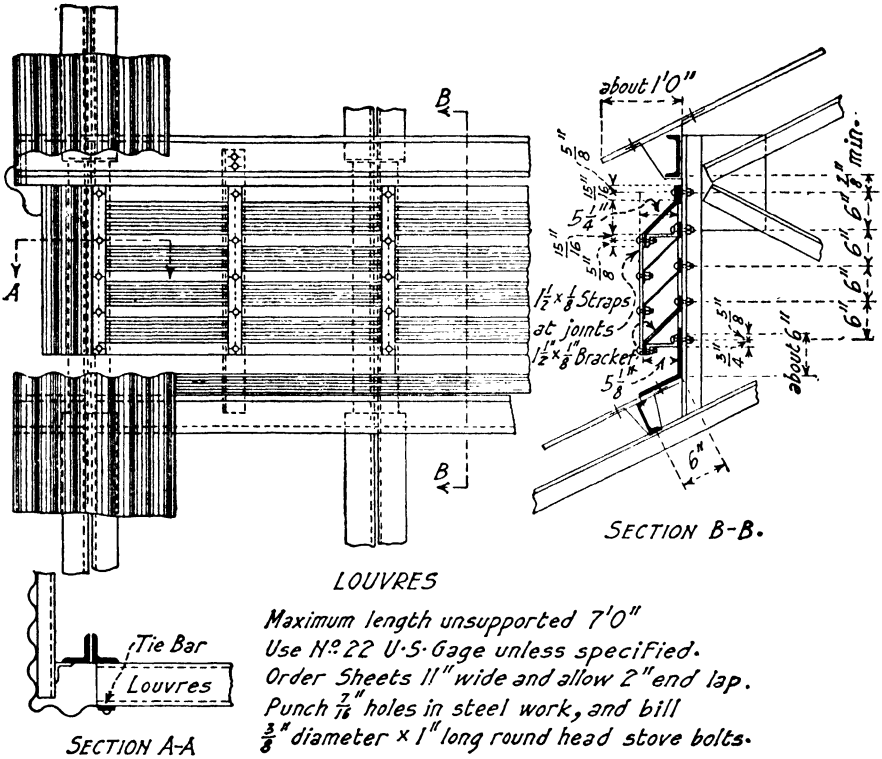


FIG. 35. DETAILS OF A STEEL MONITOR LOUVRE VENTILATOR.
AMERICAN BRIDGE COMPANY.

VENTILATORS.—Industrial buildings are ventilated either by forced draft or by natural ventilation. Natural ventilation is usually sufficient, although forced draft is necessary in many factories or mills such as cement mills and similar structures. The amount of air required depends upon the use to which the building is to be put. For mill and factory buildings it is usual to require 20 to 30 cu. ft. of fresh air per minute for each operative, and to require that all the air be entirely changed each hour. A common specification is to require a net ventilator opening per 100 sq. ft. of floor space of not less than one-fourth sq. ft. for clean machine shops and similar buildings; of not less than one sq. ft. for dirty machine shops; of not less than four sq. ft. for mills, and of not less than six sq. ft. for forge shops, foundries and smelters. The American Bridge Co. specifies that tubular ventilators shall have a net opening of one sq. ft. for each 200 to 400 sq. ft. of floor space.

Ventilators are more effective in high buildings than in low buildings. One sq. ft. of ventilator opening at a height of 60 ft. will be nearly twice as effective as one sq. ft. at a height of 20 ft.

Industrial buildings are ventilated (1) through monitor ventilators, (2) through tubular ventilators placed in the roof, or (3) by means of swing ventilators placed in the windows. The best ventilation is obtained with monitor or tubular ventilators in the roof and ventilators in the windows in the side of the building.

Details of a circular ventilator as designed by the American Bridge Company are shown in (3) Fig. 23. Details of a standard monitor steel louvre ventilator are shown in Fig. 35.

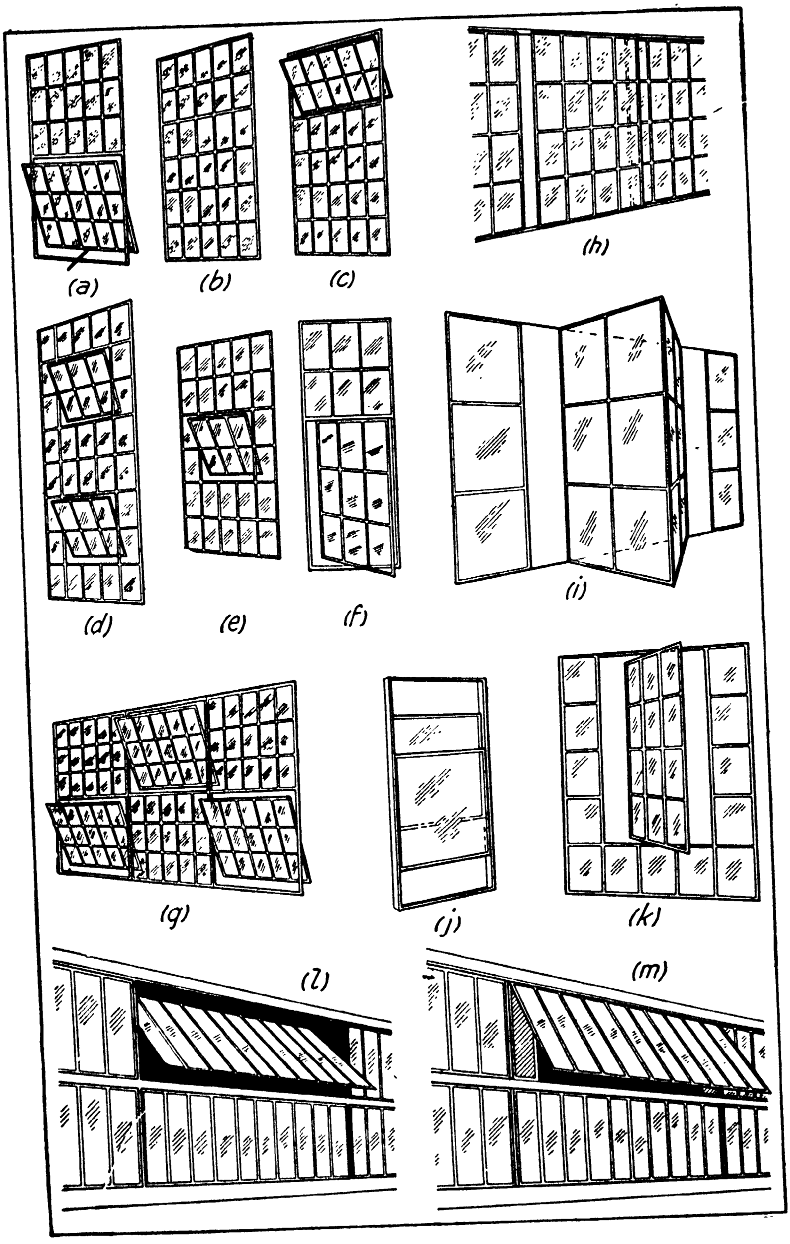


FIG. 36. TYPES OF STEEL WINDOWS.

STEEL WINDOWS.—Windows with steel sash and steel frames are now used in fireproof buildings and are generally used in all industrial buildings. The windows are generally glazed with wire glass ¼ in. thick. Window sash may be fixed, or may be opened by swinging, or by sliding horizontally or vertically.

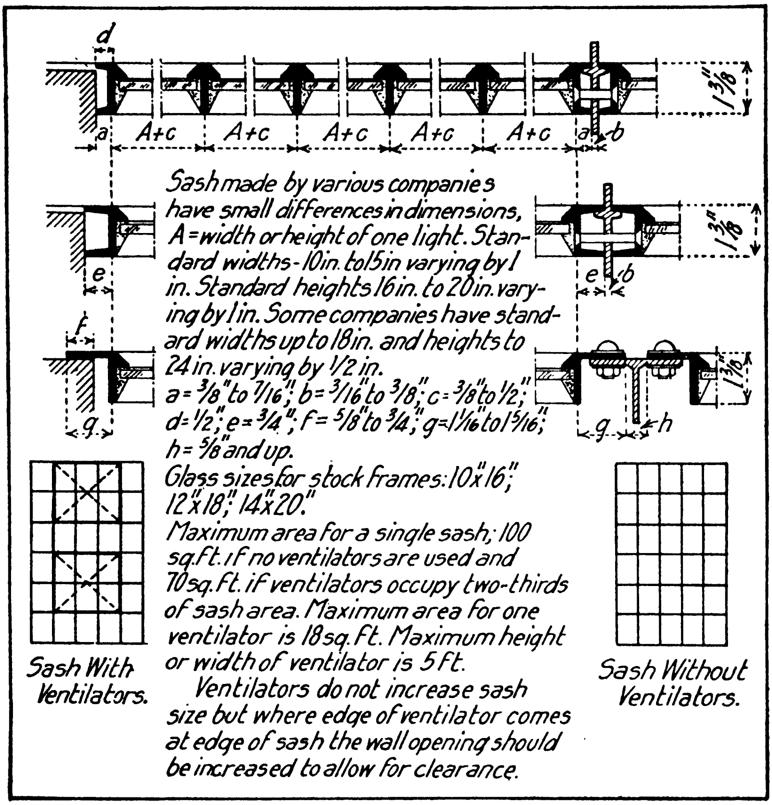


FIG. 37. STANDARD DETAILS FOR STEEL SASH.

In Fig. 36, (a) to (g) inclusive, are windows with fixed sash with ventilators in different positions; (h) is a window with horizontal sliding sash; (i) is a window with a sash which swings outward; (j) is a window with counterbalanced sash; (k) is a window with a fixed sash and a swinging ventilator; (l) is a window with a swinging sash; while (m) is a window with swinging sash with weather strips to prevent the storm from beating into the building.

Steel sash are made by many different firms. While the main dimensions of the windows made by the different firms are practically standard, each firm uses different rolled-steel sections, different details and different operating devices.

Standard dimensions for steel sash are given in Fig. 37. It should be noted that more steel is used with small sizes of glass than with large sizes, and that sash with small sizes of glass are therefore stronger than sash with large sizes. The maximum sizes of sash given in Fig. 37 are for glass 14 in. by 20 in. For glass 10 in. by 16 in. the maximum sizes may be increased 15 per cent; while for glass 18 in. by 24 in. the maximum sizes should be reduced by 15 per cent, and proportional for intermediate sizes of glass. The glass are fastened with clips and are glazed with special putty, on the inside of the sash.

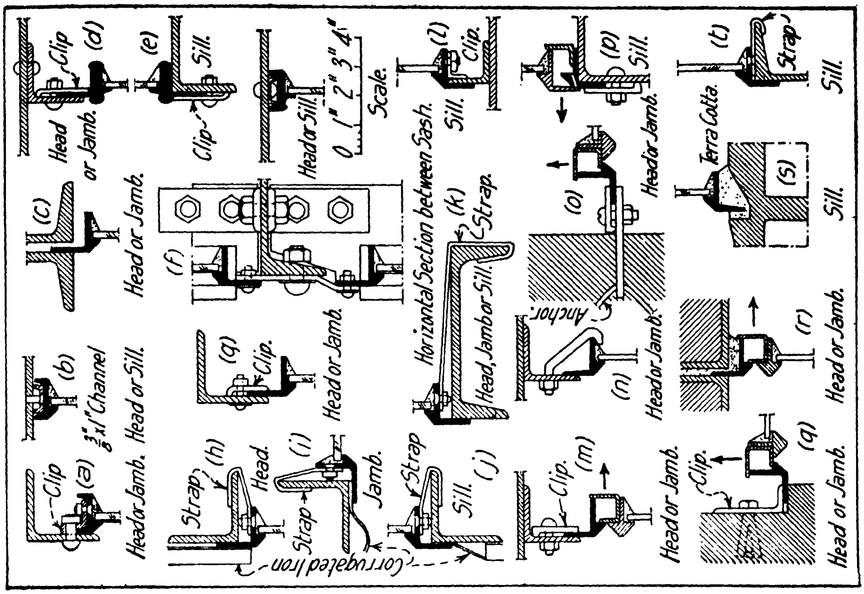


FIG. 39. DETAILS.

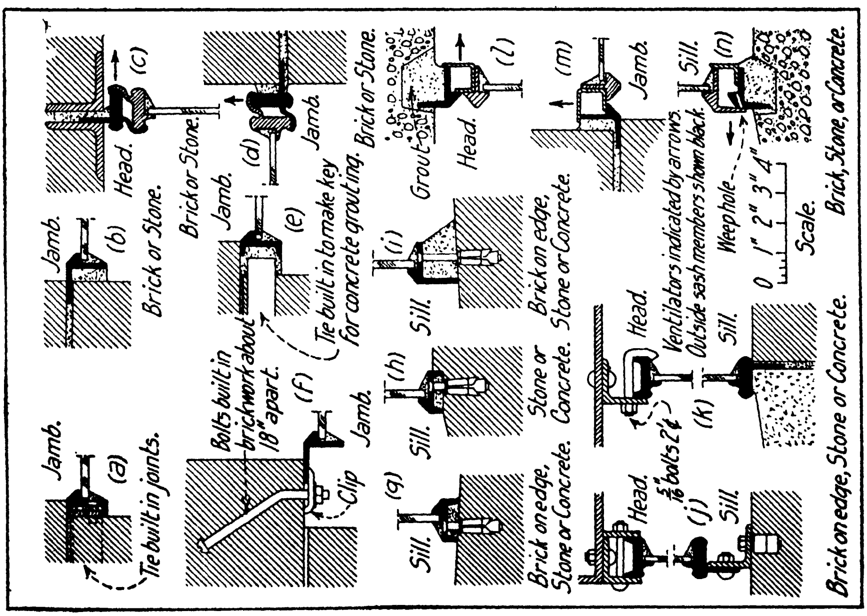


FIG. 38. DETAILS.

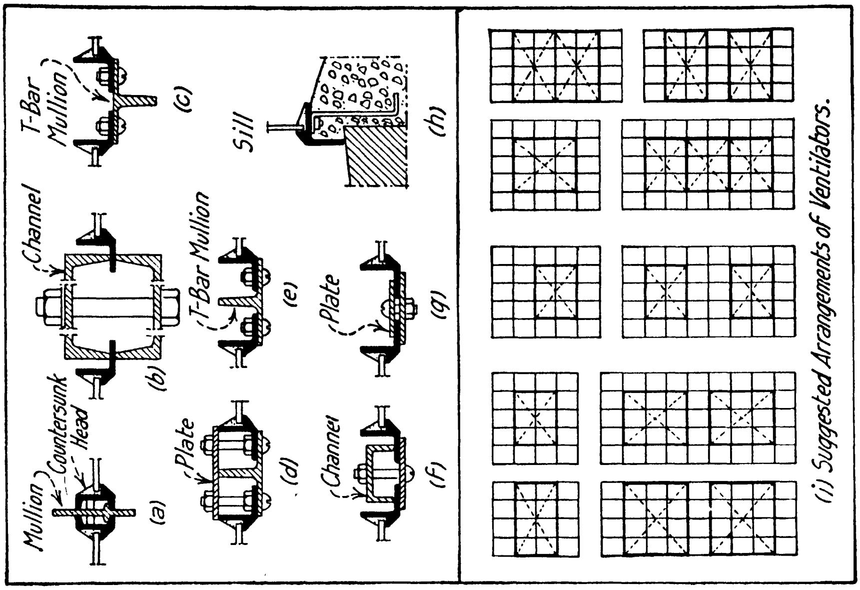


FIG. 41.

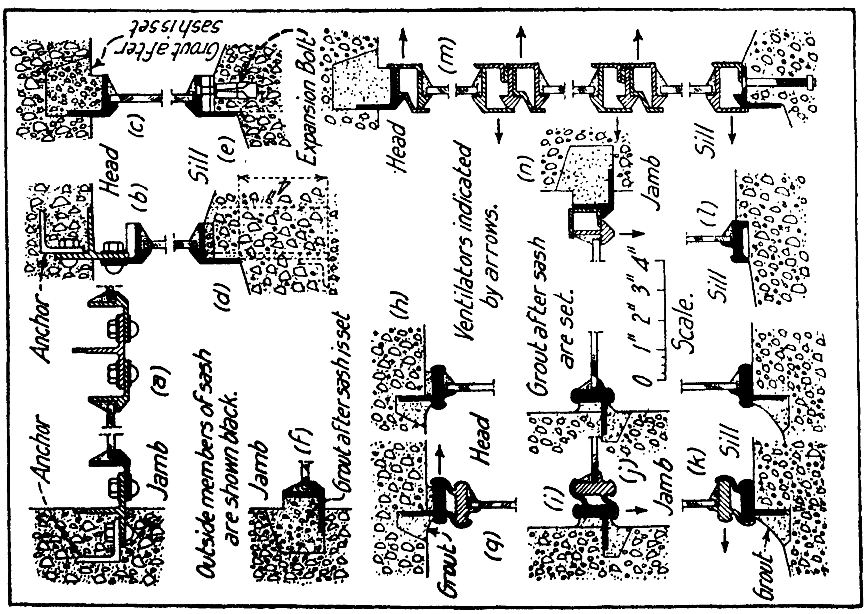


FIG. 40. DETAILS OF "UNITED STEEL SASH."

Details of window sash as taken from the catalogs of the "Fenestra" windows, made by the Detroit Steel Products Company, Detroit, Mich.; the "Lupton" windows, made by the David

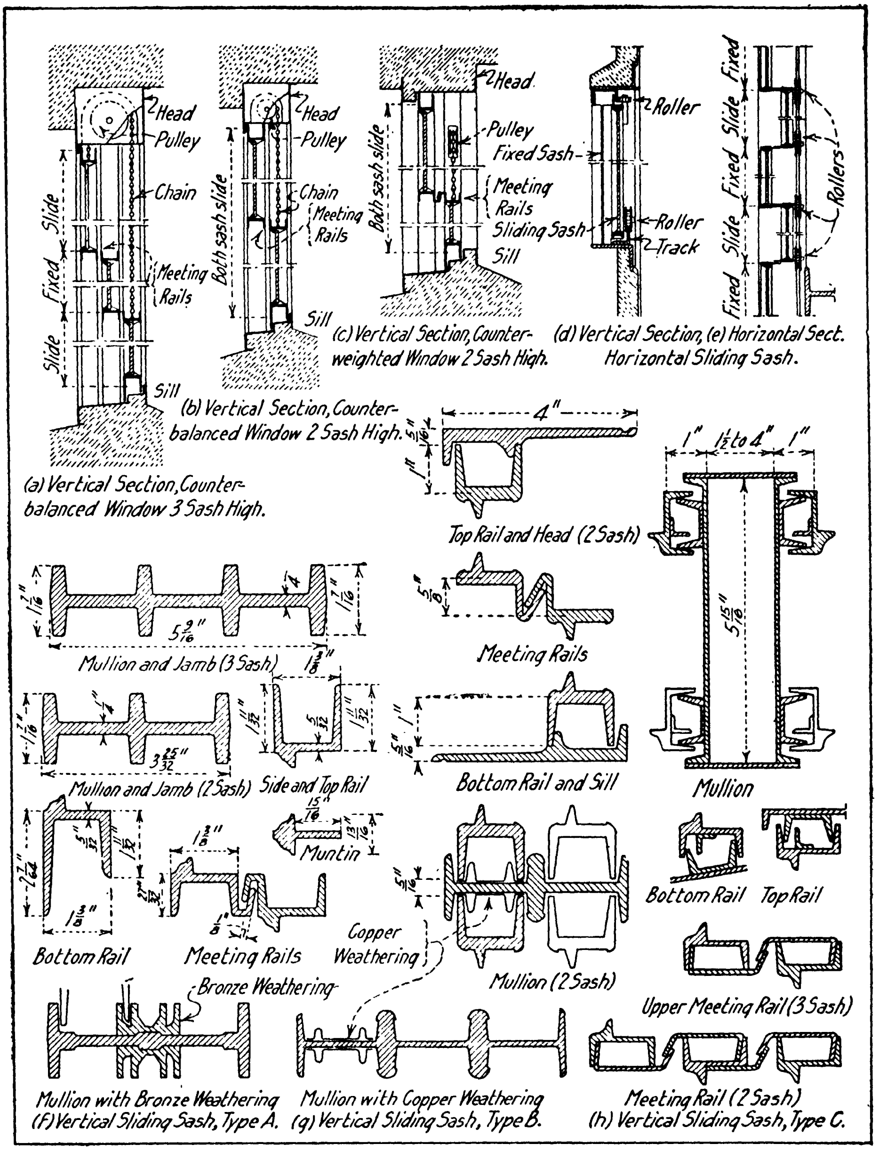


FIG. 42. DETAILS OF STEEL SASH.
((f) is "Lupton," (g) is "United Steel Sash," and (h) is "Fenestra")

Lupton Son Company, Philadelphia, and United Steel Sash" made by the Trussed Concrete Steel Co., Youngstown, Ohio, are shown in Fig. 38 to Fig. 41. While each company uses different rolled sections the details are essentially the same and may be used interchangeably as far as the

designing engineer is concerned. Details of counterbalanced sash, are shown in (a) to (c) and details of a horizontal sliding sash are shown in (d) and (e), Fig. 42. The details of the sections used by the different firms may be determined by observing that in Fig. 42 (f) is "Lupton" (g) is "United Steel Sash," and (h) is "Fenestra." Details of construction, and details of operating

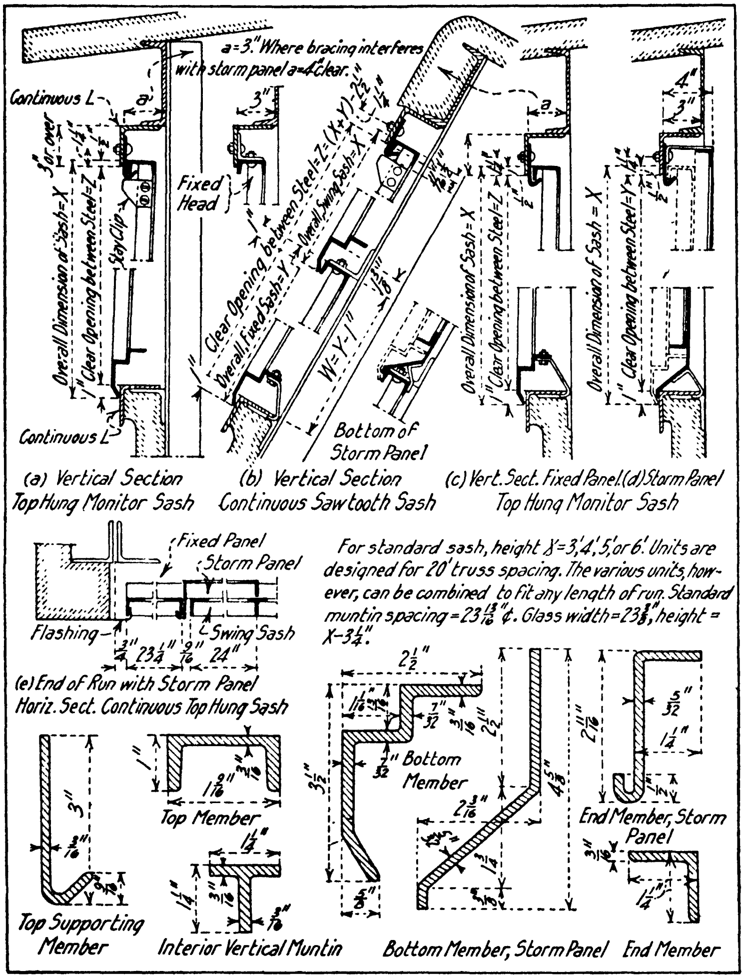


FIG. 43. DETAILS OF "UNITED STEEL SASH" VENTILATORS AND SKYLIGHTS.

devices and hardware can be obtained from the various catalogs. Details of "United Steel Sash" monitor ventilators and skylights are shown in Fig. 43. Details of "Lupton" monitor ventilators and skylights are shown in Fig. 44. The details shown in Fig. 43 and Fig. 44 are very complete. For address of other companies manufacturing steel windows, see Sweet's "Architectural Catalog" published by Sweet's Catalog Service, New York.

WOODEN DOORS.—Wooden doors are usually constructed of matched pine sheathing nailed to a wooden frame as shown in Fig. 45. These doors are made of white pine. Doors up

to four feet in width should be swung on hinges; wider doors should be made to slide on a overhead track or should be counter-balanced and raise vertically. Sliding doors should be at least 4 in. wider and 2 in. higher than the clear opening.

"Sandwich" doors are made by covering a wooden frame with flat or corrugated steel. The wooden framework of these doors is commonly made of two or more thicknesses of ⅞ in.

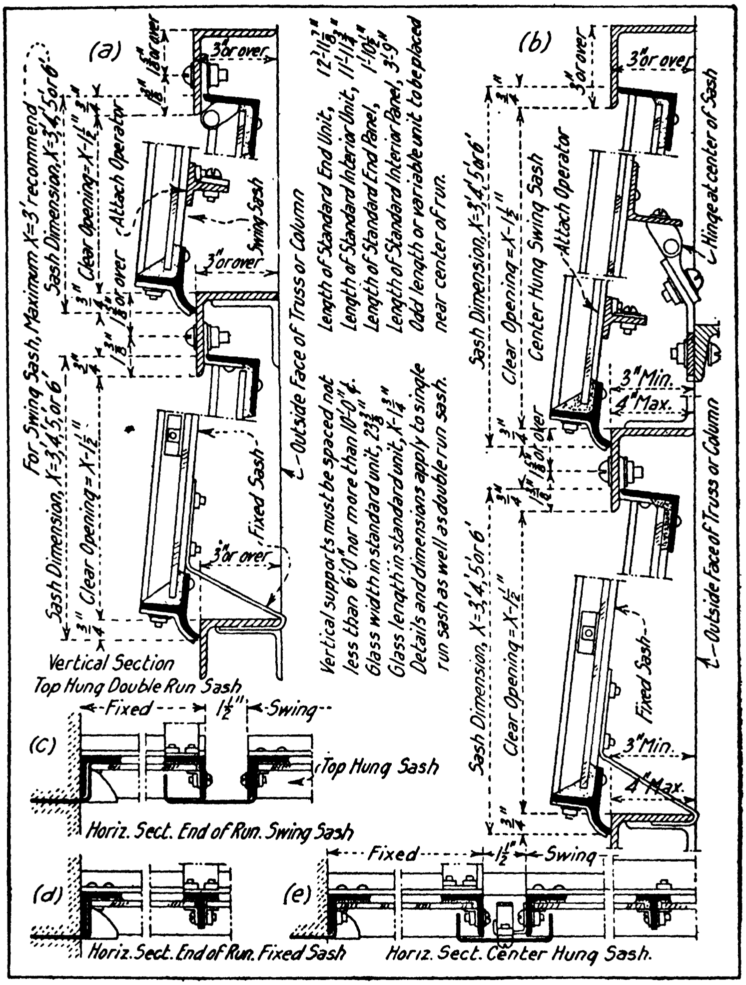


FIG. 44. DETAILS OF "LUPTON" STEEL MONITOR VENTILATORS AND SKYLIGHTS.

dressed and matched white pine sheathing not over 4 in. wide, laid diagonally and nailed with clinch nails. Care must be used in handling sandwich doors made as above or they will warp out of shape. Corrugated steel with 1¼ in. corrugations makes the neatest covering for sandwich doors.

For swing doors use hinges about as follows: For doors 3 ft. × 6 ft. or less use 10 in. strap or 10 in. T-hinges; for doors 3 ft. × 6 ft. to 3 ft. × 8 ft. use 16 in. strap or 16 in. T-hinges; for doors 3 ft. × 8 ft. to 4 ft. × 10 ft. use 24 in. strap hinges.

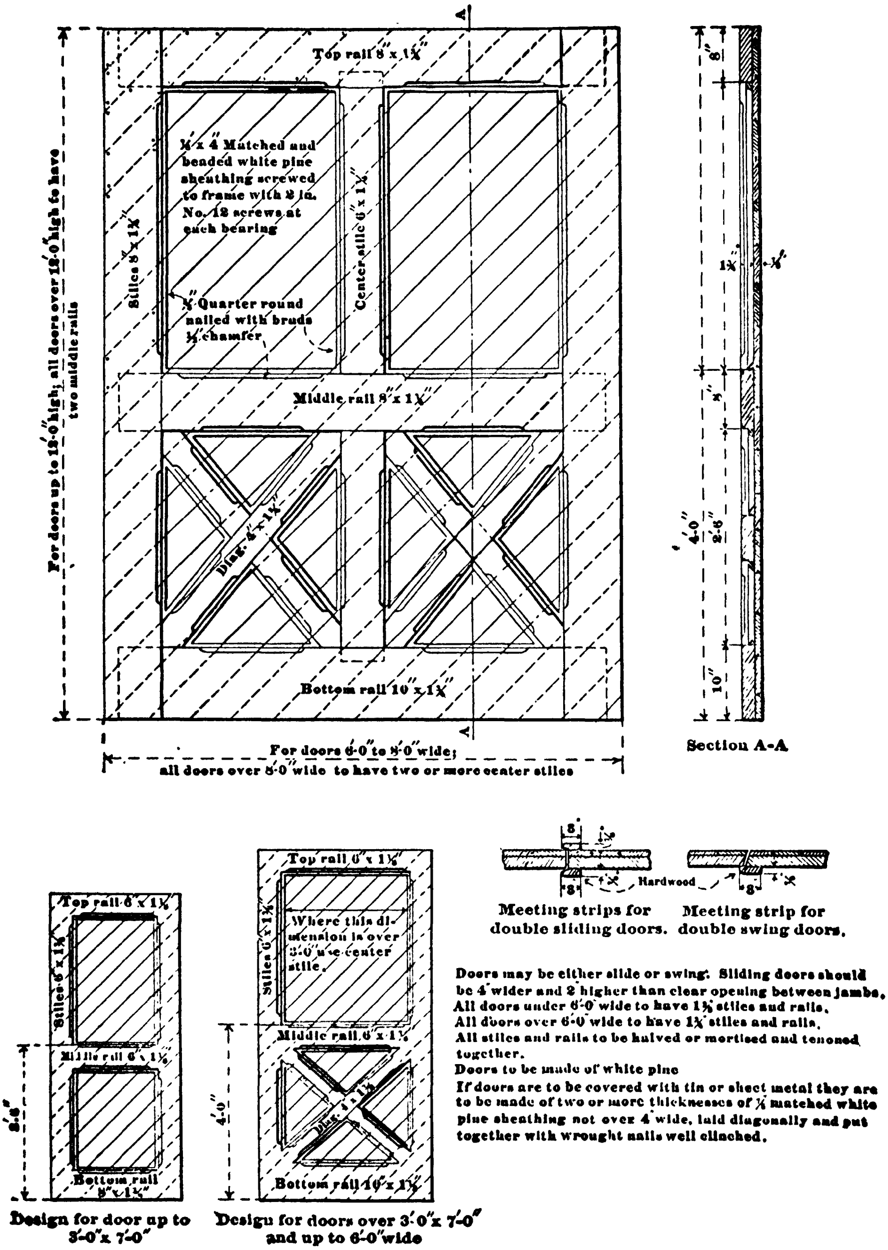


FIG. 45. DETAILS OF WOODEN DOORS. AMERICAN BRIDGE COMPANY.

STEEL DOORS.—Details of a steel sliding door are shown in Fig. 46. Details of a swinging steel door are shown in Fig. 47. Steel doors should be covered with corrugated steel, preferably with 1¼ in. corrugations.

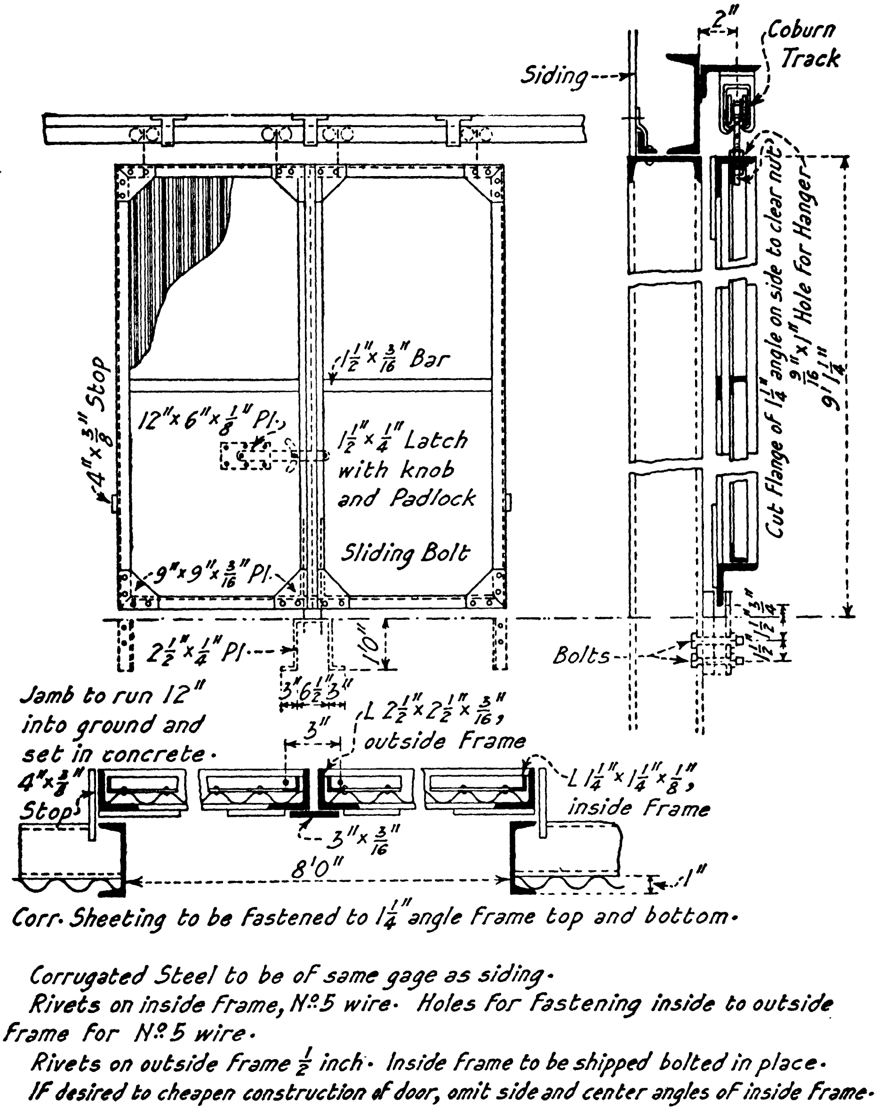
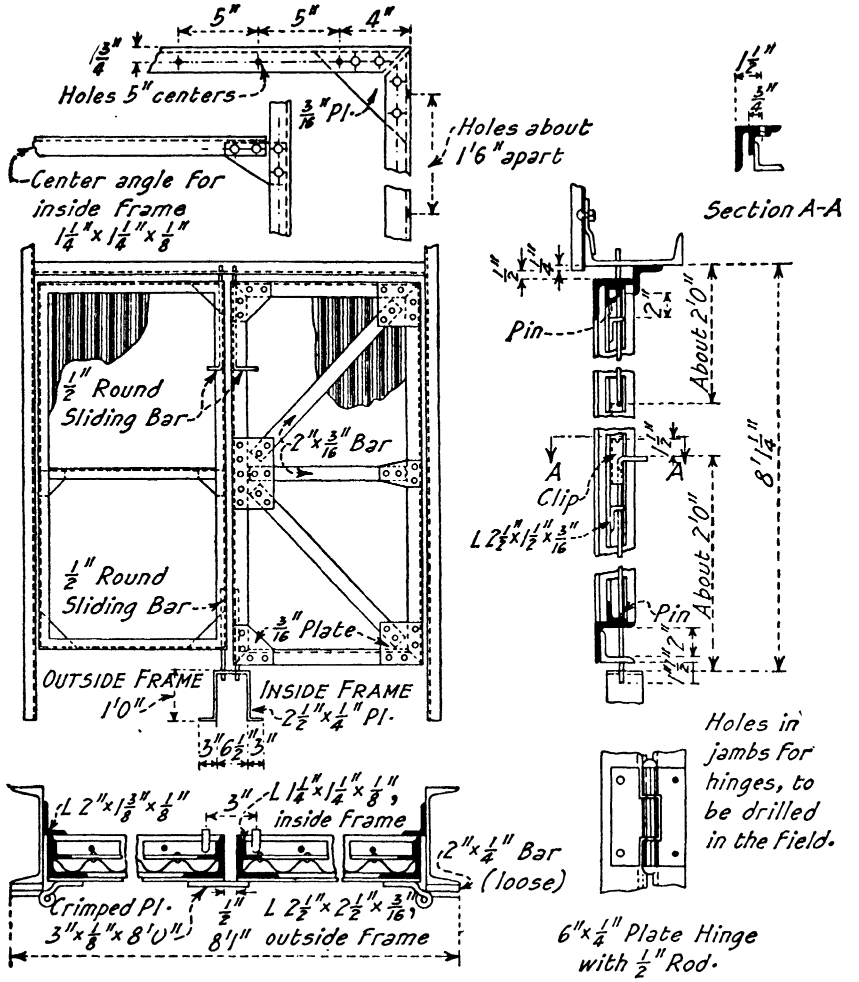


FIG. 46. DETAILS OF A SLIDING STEEL DOOR. AMERICAN BRIDGE COMPANY.

Details of the track for a sliding door are shown in Fig. 48.

Steel doors built up out of special steel sections are made by several firms. Details of " Lupton " tubular steel doors manufactured by David Lupton Sons Company, Philadelphia, Pa., are shown in Fig. 49. These doors are hinged to swing one way or slide horizontally. The



Corrugated Steel to be same gage as siding.
Rivets on inside frame, No. 5 wire. Holes for fastening inside frame to outer frame, No. 5 wire.
Rivets on outer frame $\frac{1}{2}''$ diameter. Inside frame to be shipped bolted in place.
Corrugated Steel to be riveted in field to top and bottom angles of inside frame.
If desired to cheapen construction of door, omit side and center angles of inside frame.

FIG. 47. DETAILS OF A SWINGING STEEL DOOR. AMERICAN BRIDGE COMPANY.

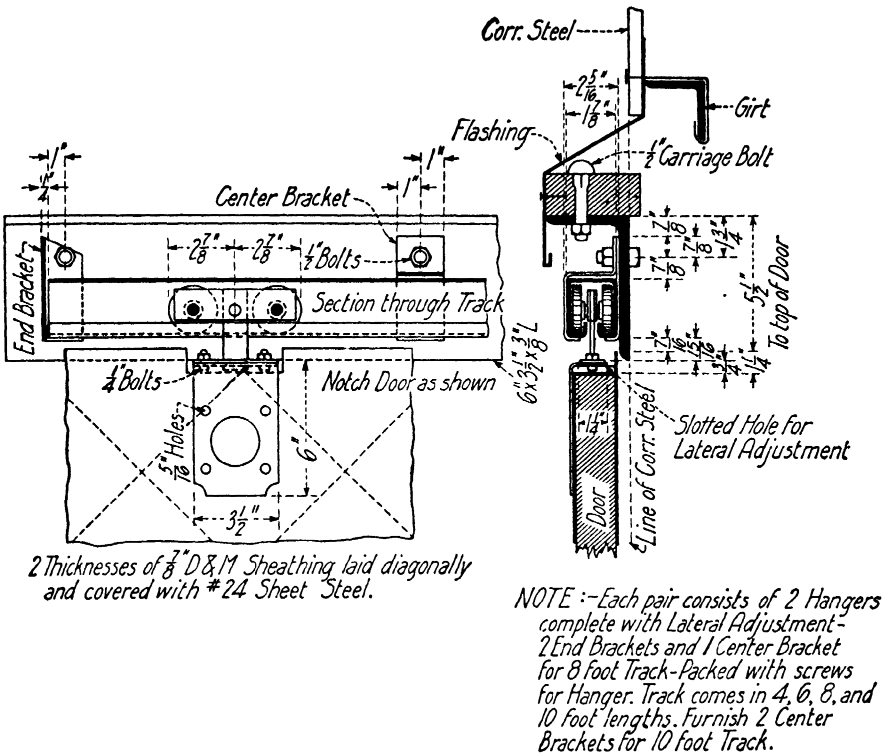


FIG. 48. DETAILS OF A TRACK FOR A SLIDING DOOR.

lower part of the door is filled with No. 12 gage steel, while the upper part is commonly filled with wire glass set in steel sash and steel frames. "Lupton" doors have the frames welded.

Details of "Fenestra" tubular steel doors made by the Detroit Steel Products Company, Detroit, Mich., are shown in Fig. 50. The doors are hinged to swing one way, or slide horizontally. Special tubular sliding doors can be made 10 ft. wide and 25 ft. high, or with double doors for an opening 20 ft. wide and 25 ft. high. "Fenestra" doors have the frames riveted. Steel doors are also made by the Trussed Steel Concrete Company.

Diagrammatic sketches of several types of doors are shown in Fig. 51. These sketches represent different types of doors shown in the catalog of J. Edward Ogden Co., New York, N. Y. This company is prepared to furnish door hardware and mechanical parts of the doors shown, or will supply the doors complete. The following data have been taken from the Ogden catalog.

Two-Section Doors.—Doors may be made of wood frame with a sheet-steel covering, or with a steel frame with a sheet-steel covering; the upper section may be glazed with ¼ in. wire glass set in metal frames. Details of doors 20 ft. wide and 22 ft. high are shown as constructed with wood frames, and also with steel frames. Counterweights are commonly made equal to one-half the total weight of the door.

Single-Section Doors.—Doors may be made with wood frames or with steel frames. Details of a door 27 ft. 9 in. wide and 19 ft. 6 in. high are shown.

Multi-Section Door.—This door is especially adapted for locations where there is little ceiling space. Doors may be made with wood frames or with steel frames. Details of doors 18 ft. 3 in. wide and 22 ft. 2 in. high are shown.

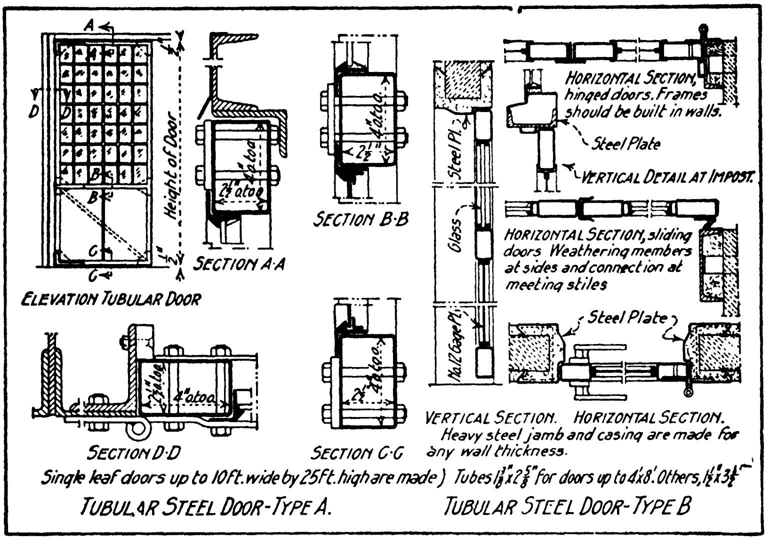


FIG. 49. DETAILS OF "LUPTON" TUBULAR STEEL DOORS.

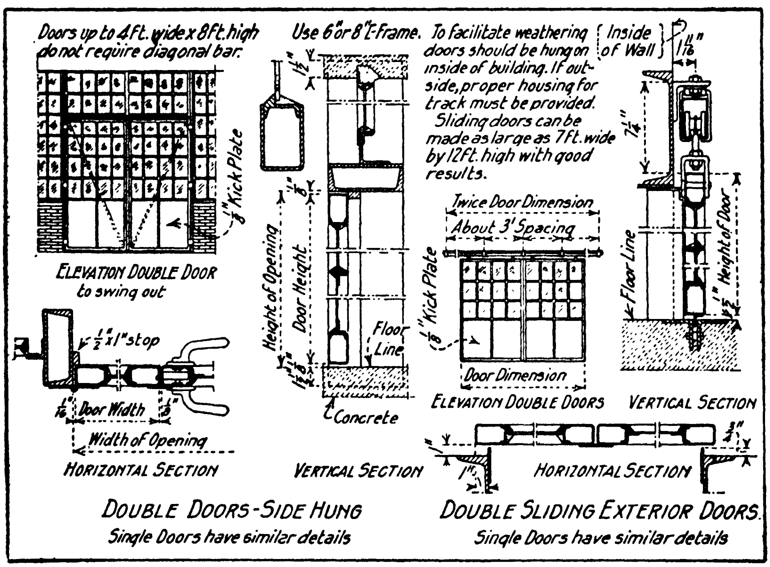


FIG. 50. DETAILS OF "FENESTRA" TUBULAR STEEL DOORS.

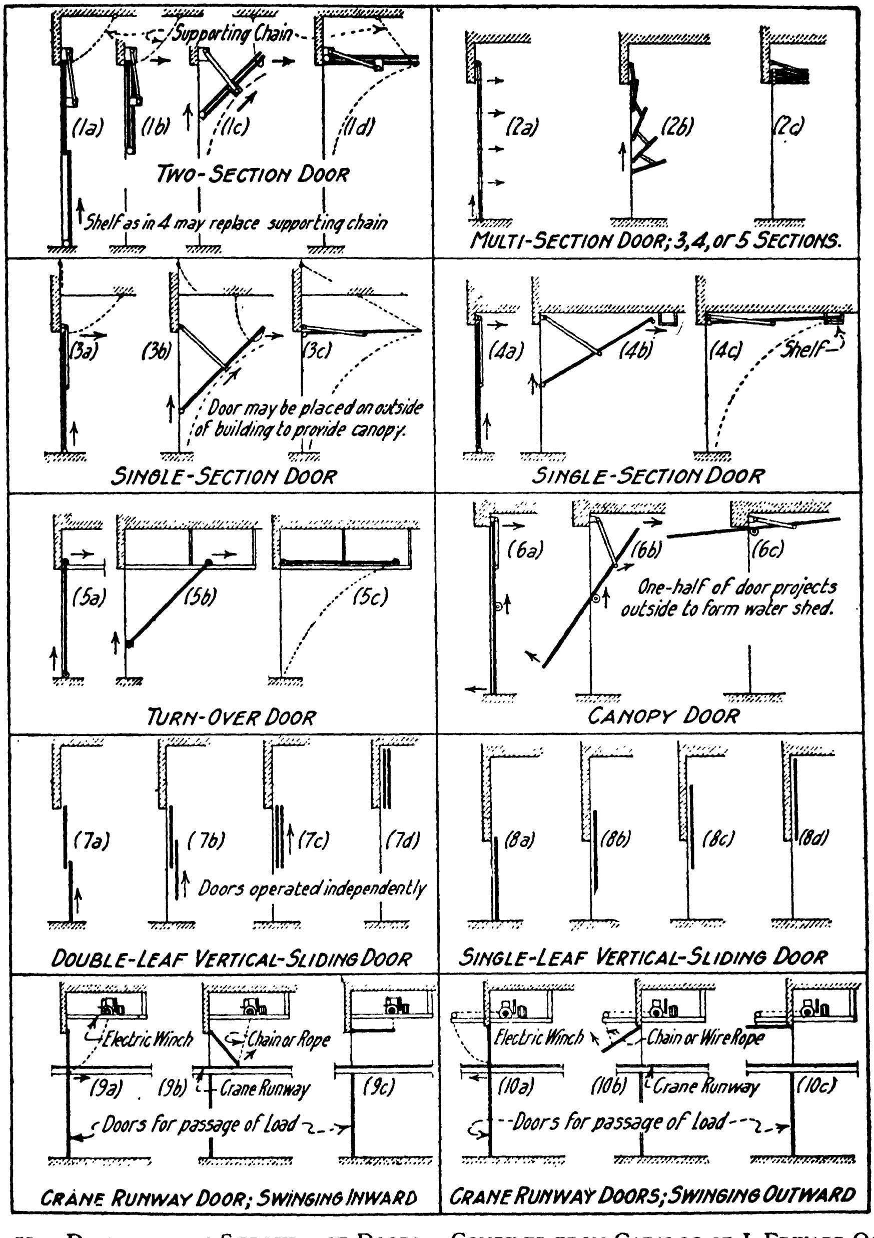


FIG. 51. DIAGRAMMATIC SKETCHES OF DOORS. COMPILED FROM CATALOG OF J. EDWARD OGDEN COMPANY.

Turn-Over Door.—This door is used for small openings. There is no operating winch, the door being operated by hand.

Canopy Door.—This door protects the entrance when open. The minimum headroom above the door is 16 inches. This is a modification of the single-section door.

Single-Leaf Vertical-Sliding Door.—These doors require adequate headroom. Details of a door 8 ft. wide and 8 ft. high are shown. These doors are often placed in pairs, where one counterweight and one winch will serve for both doors.

Double-Leaf Vertical-Sliding Doors.—The two sections of these doors are equipped with separate guides and are operated separately. Details of a door 20 ft. wide and 18 ft. high are shown.

Crane Runway Doors.—These doors may swing inward or outward. The doors may be operated by the crane operator or from the floor. Additional doors should be provided for the load, and for the crane cage where necessary.

Folding and sliding doors are also made by the Kinnear Manufacturing Company, Columbus, Ohio.

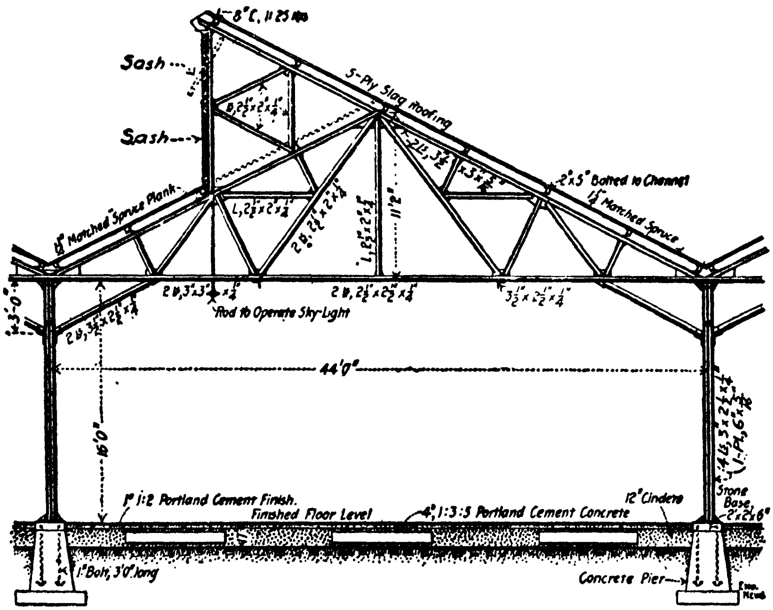
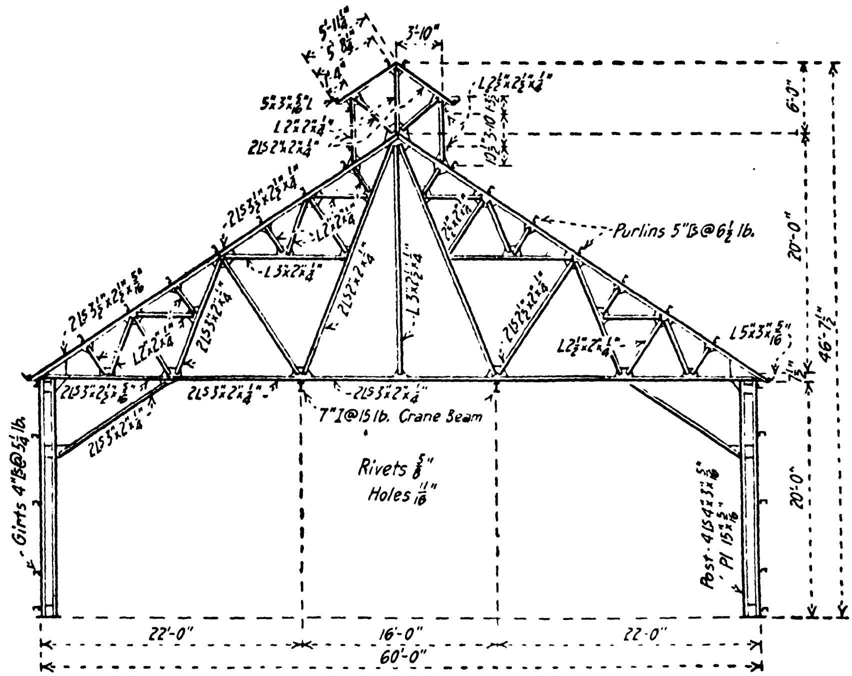


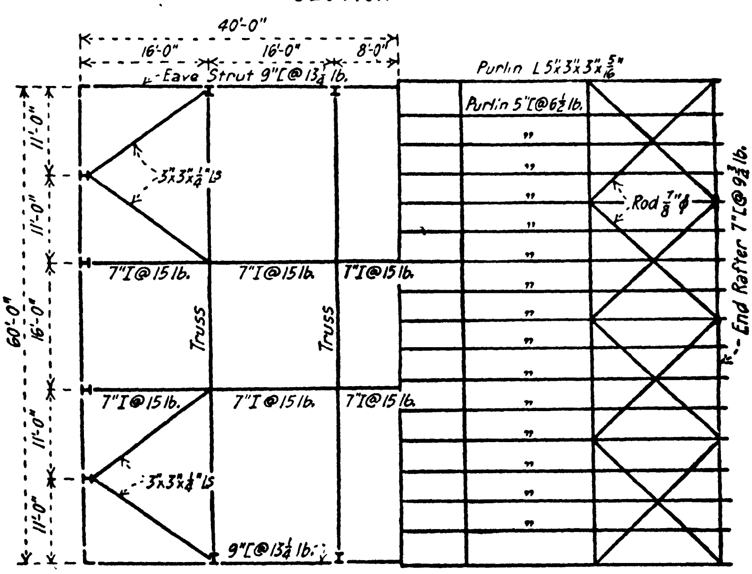
FIG. 52. MODIFIED SAW TOOTH ROOF, PAINT SHOP, PUBLIC SERVICE CORPORATION.

Rolling Steel Doors.—Rolling steel doors are made by several firms. The J. G. Wilson Corporation, New York, manufactures rolling steel doors that may be operated by hand with widths of 3 ft. to 6 ft. and heights of 6 ft. to 14 ft.; widths of 6 ft. to 10 ft. and heights of 13 ft. to 17 ft.; widths of 10 ft. to 15 ft., and heights of 13 ft. to 15 ft. Doors operated by gear have heights up to 21 ft. and widths up to 20 ft. The Kinnear Manufacturing Co., Columbus, Ohio, manufactures rolling steel doors with widths of 3 ft. to 20 ft., and heights of 6 ft. to 18 ft. For additional details and the names and addresses of other manufacturers of steel doors, see Sweet's Architectural Catalog, published by Sweet's Catalog Service, New York, N. Y.

EXAMPLES OF STEEL MILL BUILDINGS.—The following examples will illustrate the practice in the design of steel mill buildings.



SECTION



BRACING IN PLANE OF BOTTOM CHORD BRACING IN PLANE OF TOP CHORD

FIG. 53. *PLANS OF A STEEL TRANSFORMER BUILDING.

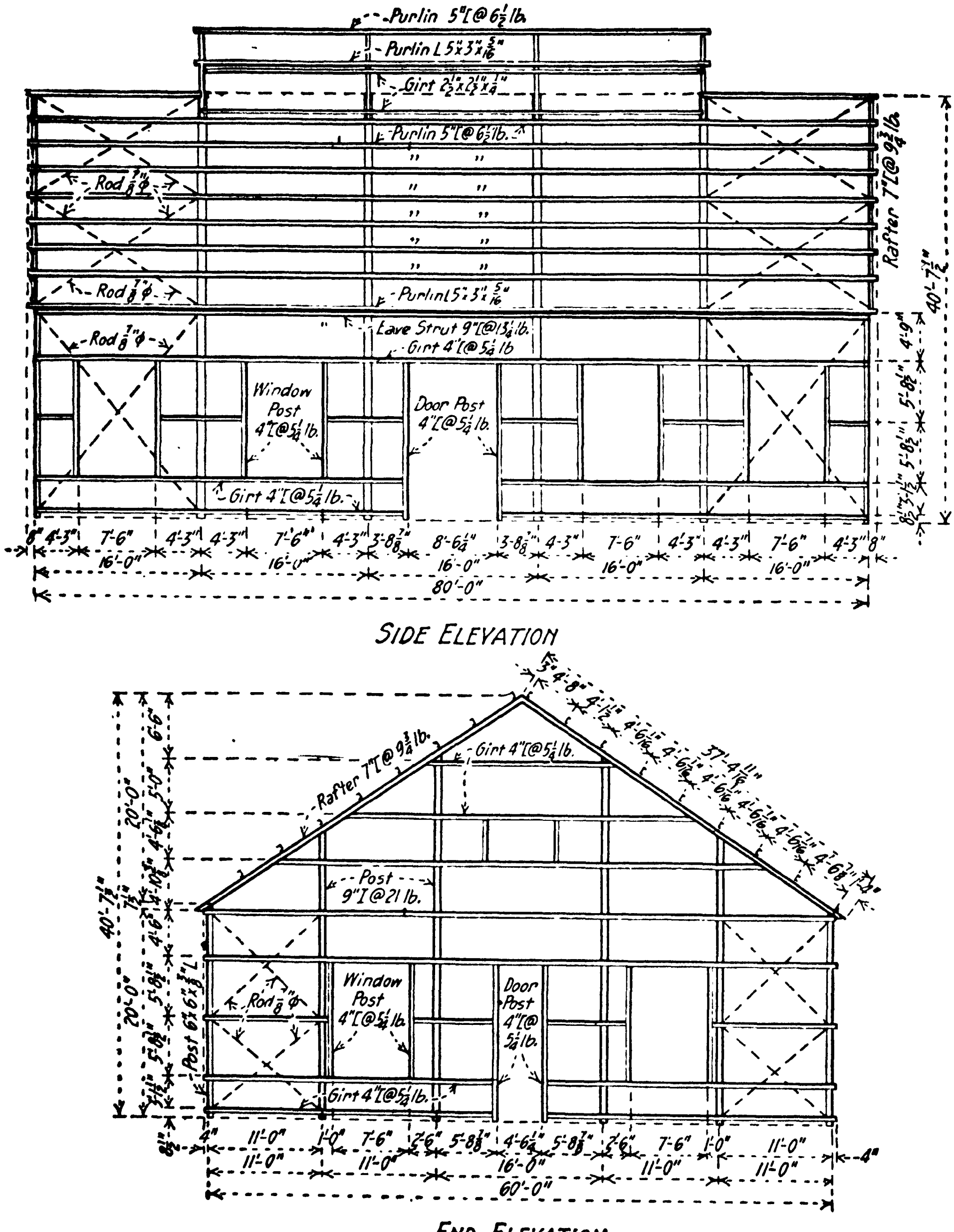


FIG. 54. PLANS OF A STEEL TRANSFORMER BUILDING.

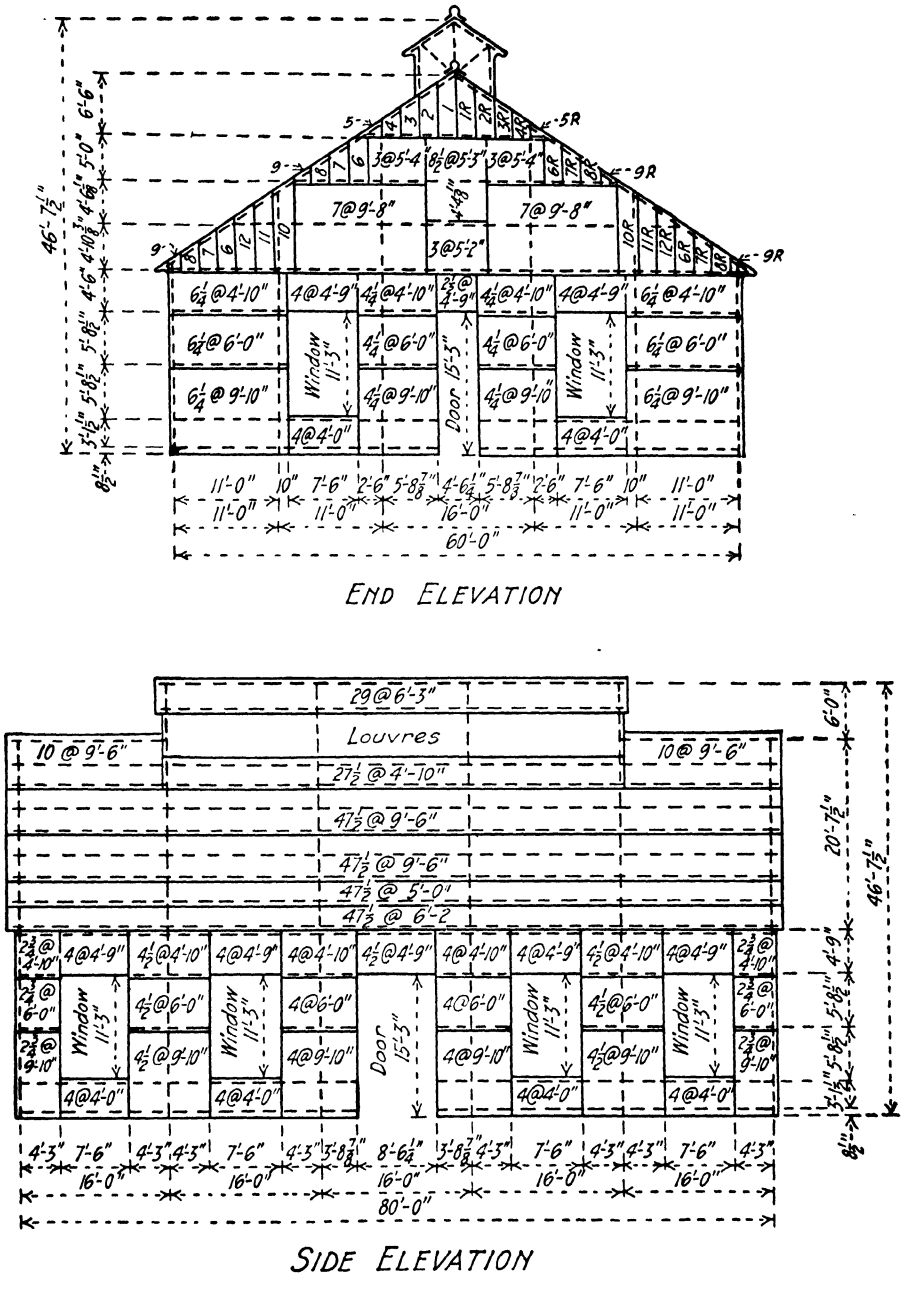
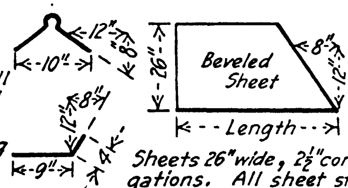


FIG. 55. CORRUGATED STEEL PLANS FOR A TRANSFORMER BUILDING.

Corrugated Steel List for Building

Rectangular Sheets				Beveled Sheets as per Sketch				
No.	U.S.S.G.	Length	Marks	No.	U.S.S.G.	Length	Marks	
55	No.22	4'-10"		4	No.24	7'-1½"	2 No.1	2 No.1R
95	"	5'-0"		4	"	5'-9½"	2 " 2	2 " 2R
95	"	6'-2"		4	"	4'-5½"	2 " 3	2 " 3R
58	"	6'-3"		4	"	3'-1½"	2 " 4	2 " 4R
40	"	9'-6"		4	"	1'-9½"	2 " 5	2 " 5R
190	"	9'-6"		8	"	6'-0"	4 " 6	4 " 6R
48	No.24	4'-0"		8	"	4'-8"	4 " 7	4 " 7R
62	"	4'-9"		8	"	3'-4"	4 " 8	4 " 8R
87	"	4'-10"		8	"	2'-0"	4 " 9	4 " 9R
7	"	5'-2"		4	"	10'-0"	2 " 10	2 " 10R
7	"	5'-3"		4	"	8'-8"	2 " 11	2 " 11R
12	"	5'-4"		4	"	7'-4"	2 " 12	2 " 12R
87	"	6'-0"						
28	"	9'-8"						
87	"	9'-10"						

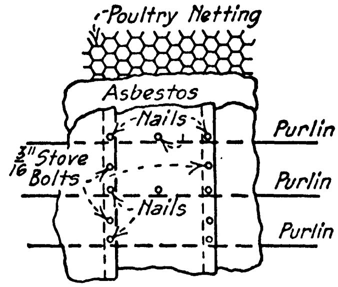


84 lin. ft. Ridge Roll No. 22 flat steel

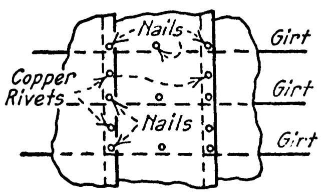
100 lin. ft. Flashing No. 22 flat steel

Sheets 26" wide, 2½" corrugations. All sheet steel painted one coat red lead.

55 squares Asbestos
1300 lin. ft. 60" Poultry Netting



METHOD OF FASTENING STEEL AND LINING ON ROOF



METHOD OF FASTENING STEEL ON THE SIDES

Corrugated steel on sides, No. 24 Black, Painted. 1 corrugation side lap and 4" end lap.
Corrugated steel on roof, No. 22 Black, Painted. 2 corrugated side lap and 6" end lap.

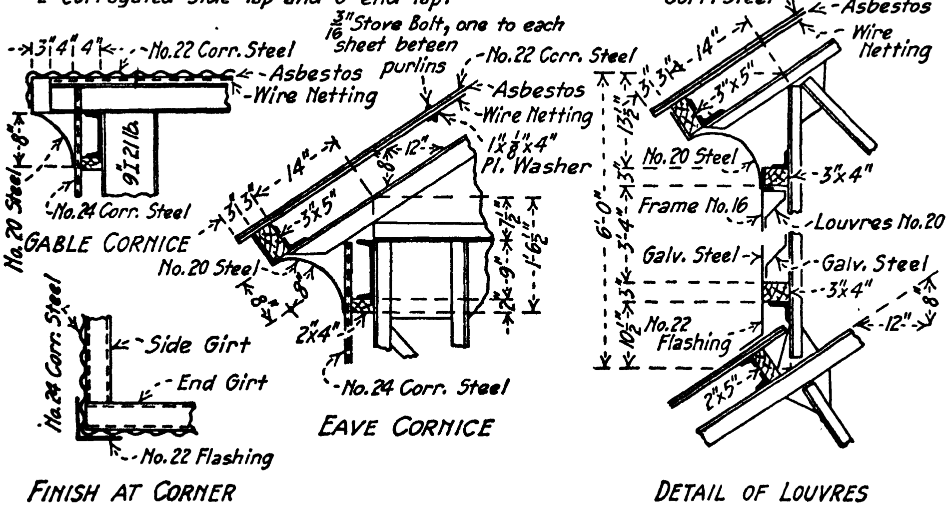


FIG. 56. CORRUGATED STEEL LIST AND DETAILS FOR TRANSFORMER BUILDING.

Example of Ketchum's Modified Saw Tooth Roof.—The modified form of saw tooth roof shown in (n) Fig. 6, was proposed by the author in the first edition of "The Design of Steel Mill Buildings" (1903). This form of saw tooth roof has been used in the paint shops of the Plank Road Shops of the Public Service Corporation of New Jersey, Newark, N. J. The building proper is 135 ft. wide by 354 ft. long. The main trusses are of the modified saw tooth type with 44 ft. spans and a rise of ¼, and are spaced 16 ft. centers. The general details of one of the main trusses are shown in Fig. 52. The building has an independent steel framing with brick curtain walls on the exterior. Pilasters 24 in. by 20 in. are placed 16 ft. apart under the ends of the trusses, the intermediate curtain walls being 12 in. thick. The roof is a 5 ply slag roof laid on tongued and grooved spruce sheathing, which is spiked to 2 in. × 5 in. spiking strips, which are bolted to 8 in. channel purlins spaced 6 ft. centers.

Ketchum's modified saw tooth roof has been used, with excellent results, in the shops and engineering laloratory buildings of the University of Colorado.

Steel Transformer Building.—The framework of a steel frame transformer building is shown in Fig. 53 and Fig. 54. The transverse bents are made of Fink trusses, knee-braced to plate and angle columns. The bents are spaced 16 ft. centers. The members of the truss are made of angles placed back to back, the members being riveted to connection plates. The main columns are I-shaped, each flange being composed of two angles placed back to back, with the long legs outstanding and fastened together with a web plate. The columns in the ends of the building are made of 9-in. I beams. The main purlins are made of 5-in. [s @ 6½ lb., while the girts are 4-in. [s @ 5¼ lb. The purlins are spaced less than 4 ft. 9 in., which is a maximum spacing where corrugated steel roofing is used without sheathing. The steel framework is braced in the plane of the top chord and in the sides and ends of the building by means of diagonal rods 7/8 in. in diameter. The crane girder beams in the plane of the lower chord, together with the diagonal bracing, braces the building longitudinally. The diagonal bracing in the plane of the lower chord is made of angles.

The plans for the corrugated steel covering on the roof and sides of the building are shown in Fig. 55 and Fig. 56. The corrugated steel for the roof is No. 22 gage steel with 2½-in. corrugations, while the corrugated steel for the sides is No. 24 gage steel with 2½-in. corrugations. The flashing and ridge roll are made of No. 22 flat sheet steel. The finish of the building at the corners, and the eave and gable cornice are shown in Fig. 56.

To prevent the condensation of moisture on the inside of the steel roof and the resulting dripping, anti-condensation lining was used, as shown in Fig. 56. This lining was constructed as follows: Galvanized wire poultry netting was fastened to one eave purlin, was passed over the ridge, stretched tight and fastened to the other eave purlin. The edges of the wire were woven together by means of wire clips. On the wire netting was laid two layers of asbestos paper, 1/16 in. thick, and on top of the asbestos was laid two layers of tar paper. The corrugated steel was then laid on the roof in the usual way and was fastened to the purlins by means of long, soft iron wire nails, placed as shown in Fig. 56. To prevent sagging of the lining, stove bolts 3/16 in. in diameter, with 1 in. × 1/8 in. × 4 in. flat washers on the lower side, were placed between the purlins. Where anti-condensation lining is used, better results will be obtained if the purlins are spaced one-half the usual distance, in which case the stove bolts may be omitted.

Steel Frame Building with Plaster Walls.—The steel frame building shown in Fig. 57 was covered with expanded metal and plaster walls and roof constructed as follows: The side walls were made by fastening ¾ in. channels at 12 in. centers to the steel framework and then covering this framework with expanded metal wired on. The expanded metal was then covered on the outside with a coating of cement mortar composed of one part Portland cement and two parts sand, and on the inside with a gypsum plaster, making the walls about 2 in. thick. The roof consists of a 2½ in. concrete slab reinforced with expanded metal, this slab being covered with 10 in. × 12 in. slate nailed directly to the concrete.

Machine Shop, U. S. Government Powder Plant, Nitro, West Virginia.—The steel framework for the machine shop erected at the U. S. Government Powder Plant, Nitro, West Virginia,

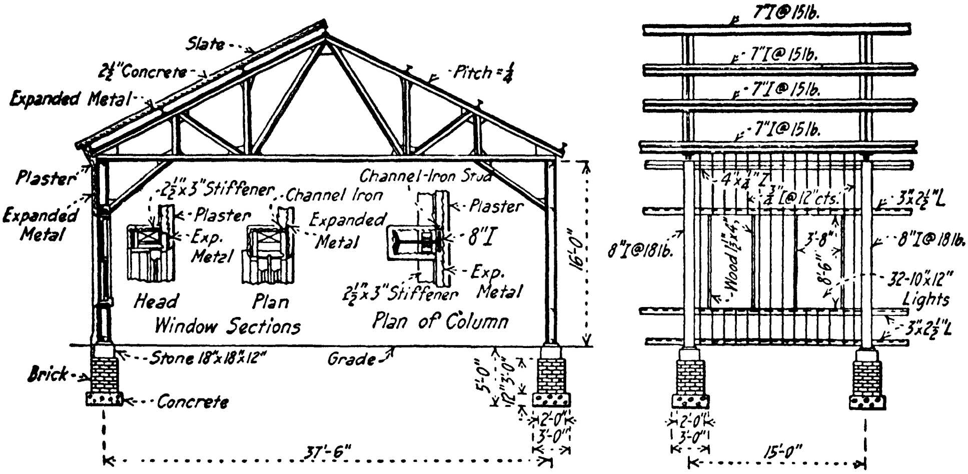


FIG. 57. STEEL FRAME BUILDING WITH PLASTER WALLS.

is shown in Fig. 58. The building is 100 ft. wide and 200 ft. long. The main truss spans are 51 ft. 4 in., with a distance of 33 ft. to the bottom chord of the truss. The side sheds have a span of 24 ft. 4 in. The roof has a slope of 5 in. in 12 inches. The transverse bents are spaced 20 ft. centers and are braced as shown in Fig. 10. The main columns are made of one 15-in. I @ 38 lb. (Bethlehem) and one 10-in. [@ 15 lb. The side columns are made of one 8-in. I @ 18 lb., while the end columns are made of one 12-in. I @ 31½ lb. The main columns are spaced 40 ft. centers. The intermediate transverse bents are carried on longitudinal trusses carried on the main columns. These longitudinal trusses carry the 10-ton crane and also act as longitudinal braces for the building. The roof is made of No. 22 corrugated steel laid on 2-in. yellow pine sheathing. The sides are covered with No. 22 corrugated steel fastened directly to the girts. An 8-in. brick wall, 4 ft. high, is built between the columns. The skylights and windows are made of Fenestra steel sash, with 10 in. × 16 in. lights, and are glazed with ¼-in. wire glass. The window sills of the lower windows in the sides of the building are on the top of the brick wall. The building is ventilated through the Fenestra sash, as shown in Fig. 58. The building is well lighted, 23 per cent of the total exterior surface of the building being glazed, while 60 per cent of the side walls are glazed.

The floor was made of 3-in. creosoted timber blocks laid on a 6-in. concrete base. Creosoted blocks were laid on a layer of 1 : 4 Portland cement mortar, ½ in. thick. The joints were filled with bituminous material. Expansion joints 1 in. thick were made around all columns and around all exterior walls to provide for expansion.

For an estimate of the weight and the cost of this building, see the author's "Design of Steel Mill Buildings," Fourth Edition.

Steam Engineering Building.—Details of a transverse bent of the steam engineering building at the Brooklyn Navy Yard are given in Fig. 59.

The main columns are spaced 48 ft. centers while the main trusses are spaced 16 ft. centers. The intermediate trusses are carried on heavy trusses rigidly fastened to the main columns. The crane girders are carried on crane columns that are fastened to the main columns by light lacing. This method of supporting heavy crane girders is the most satisfactory method yet proposed. The building is well lighted with glass in the side walls, and sky lights in the roof. More than 60 per cent of the area of the external walls and roof is glazed. Many other interesting details can be obtained from the drawings.

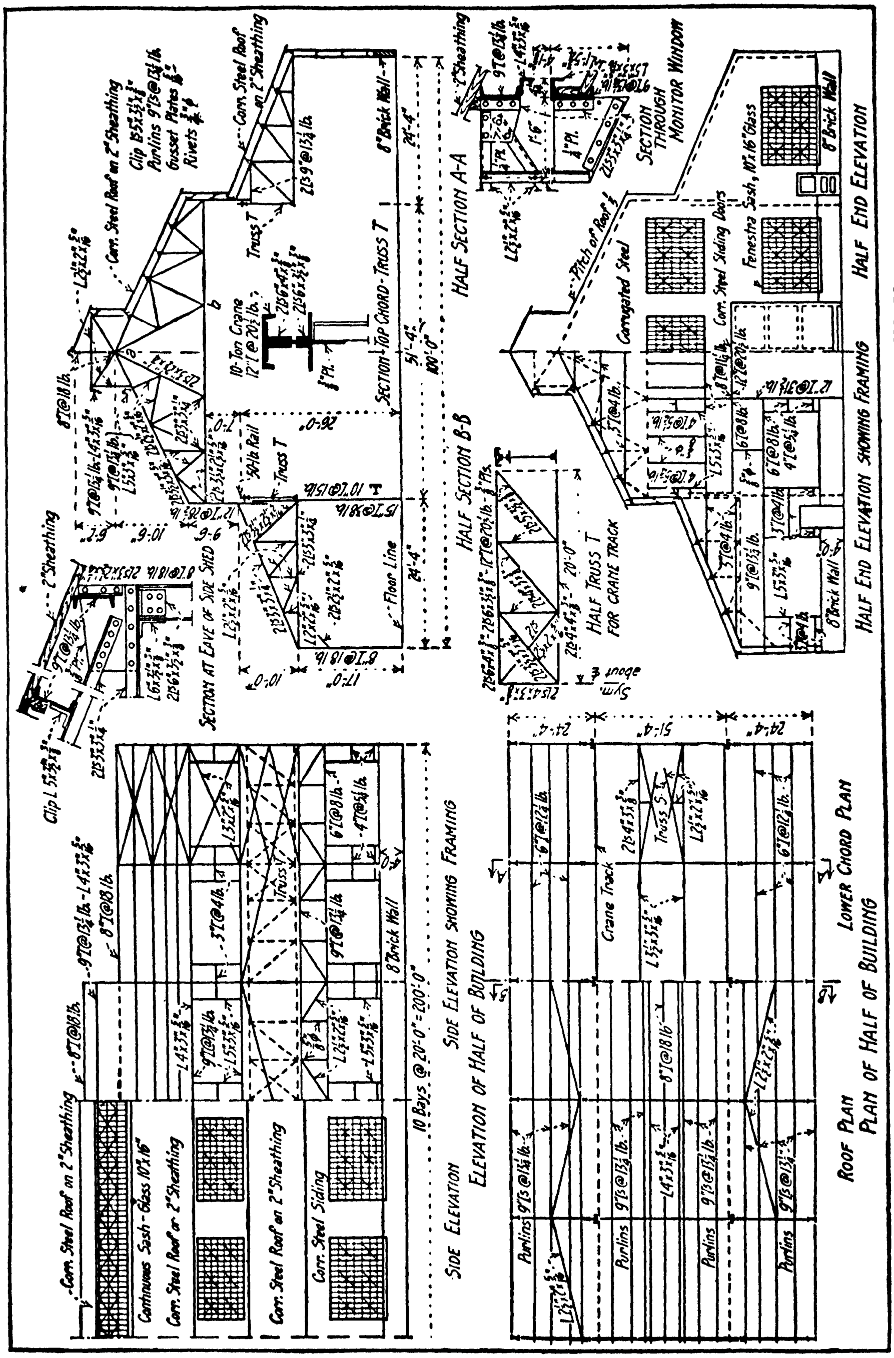


FIG. 58. MACHINE SHOP, U. S. GOVERNMENT POWDER PLANT, NITRO, W. VA.

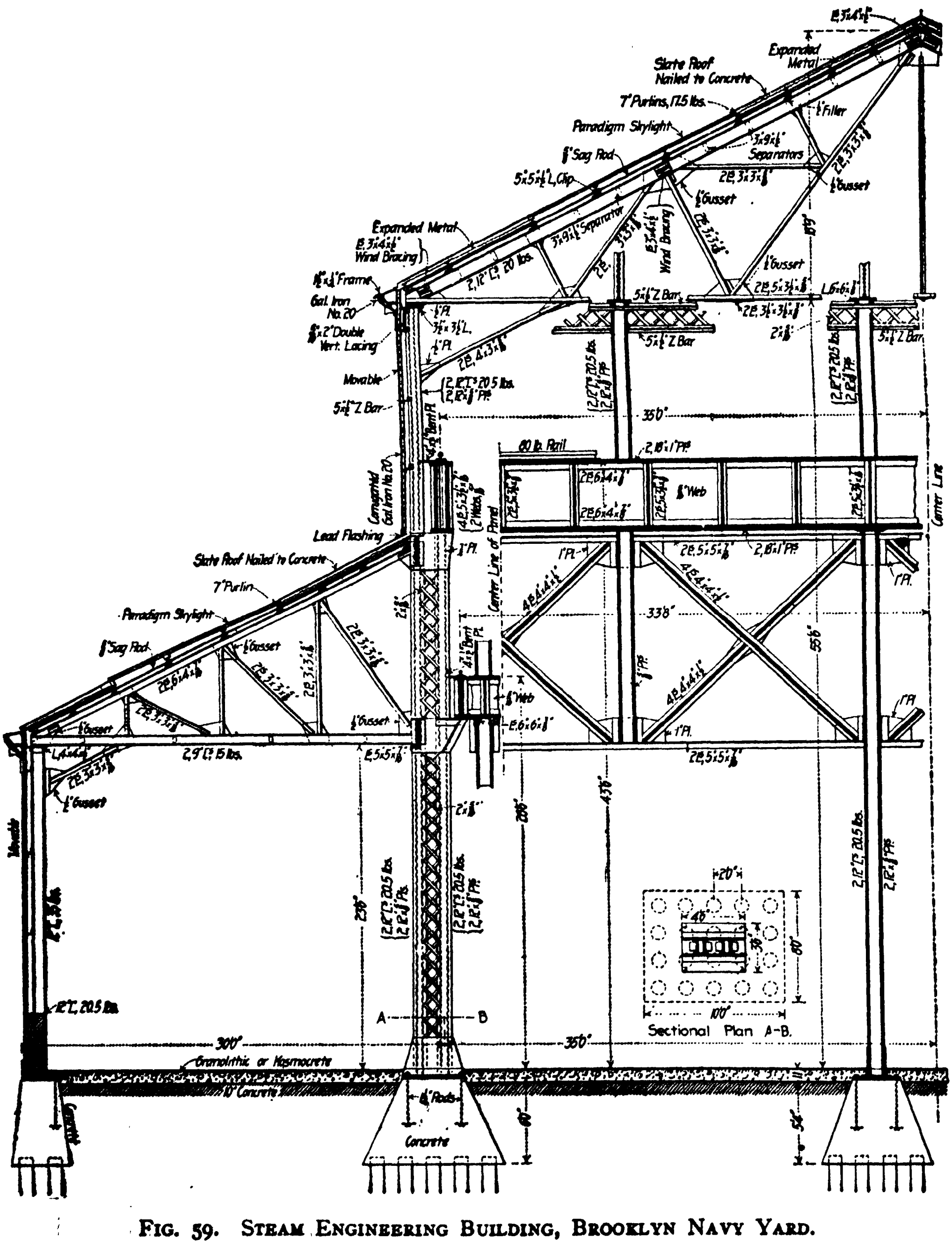


FIG. 59. STEAM ENGINEERING BUILDING, BROOKLYN NAVY YARD.

STRESSES IN MILL BUILDING COLUMNS CARRYING CRANE LOADS.—The stresses produced in columns of mill buildings by crane loads eccentrically applied depend upon the method used in bracing the structure against lateral forces. If the kneebraces are omitted or only very small kneebraces are used, the columns are practically hinged at the top and the lateral thrust due to the eccentric crane loads must be carried to the ends of the building by the lateral bracing in the planes of the chords of the trusses. Proper bracing must then be provided in the end bents.

If rigid kneebraces are provided the columns may be considered as fixed at the top and a transverse bent may be considered as carrying its load directly to the foundations. The lateral load will in reality be distributed between the direct path down the columns and the indirect path along the lateral bracing in the planes of the chords to the end bents. The portion carried by each route will depend upon the relative rigidity of the routes. Since the transverse bent is much more rigid than the lateral bracing, all of the load may be considered as carried by the transverse bent.

In Fig. 60 three cases are considered.

Case I. Columns Hinged at Base and Top.—This case is statically determinate. The lateral thrust is taken by the bracing in the plane of the chords and by the bracing in the end bents.

Case II. Columns Hinged at Base and Fixed at Top.—Columns with constant cross-section.—The formulas for rigid frames were used, making the ratio of the moment of inertia of the truss to the moment of inertia of the column equal to infinity. The formula is sufficiently accurate when this ratio becomes as small as four, and is on the safe side. The distance h is measured to a point one-half way between the foot of the knee-brace and the top of the column.

Case III. Columns Hinged at the Base and Fixed at Top. Columns with variable cross-sections.—In this case the column has a different cross-section above and below the attachment

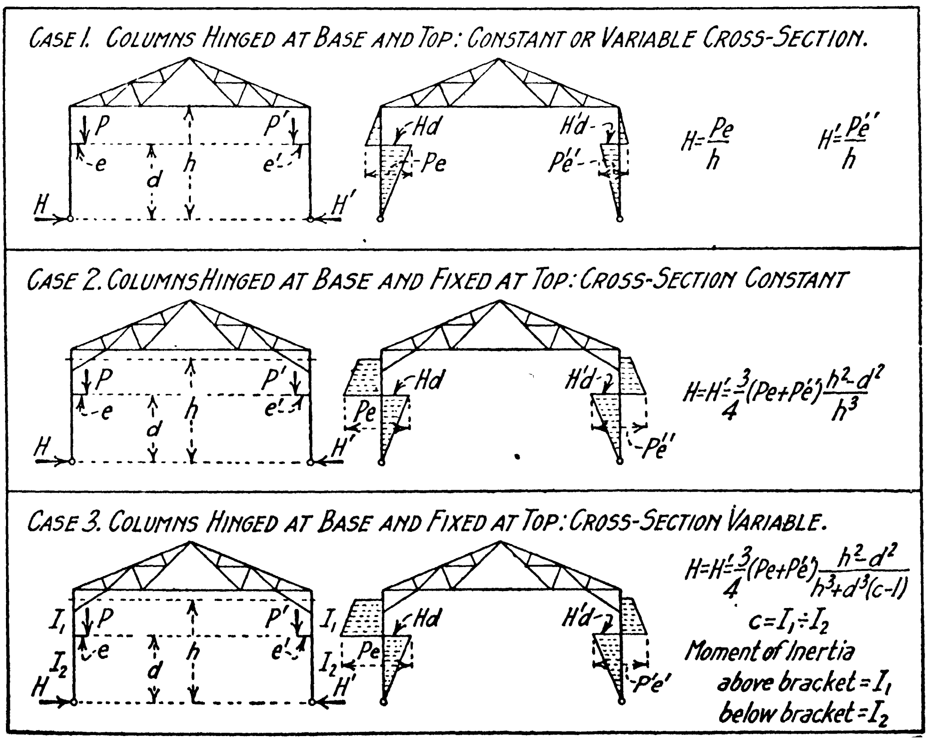


FIG. 60. STRESSES IN MILL BUILDING COLUMNS CARRYING CRANE LOADS.

of the crane girder. The formulas for rigid frames were used, making the ratio of the moment of inertia of the truss to the moment of inertia of the column equal to infinity. The formula is sufficiently accurate with a ratio of four and is on the safe side.

Case IV. Columns Fixed at Base and Fixed at Top.—Formulas for Case II and Case III may be used, the value of h being taken as the distance from the point of contraflexure to a point midway between the foot of the kneebrace and the top of the column. The point of contraflexure may be calculated by formula (4), Chapter XVI.

Stresses in Rigid Frames.—Formulas for stresses in rigid frames with pin-connected columns, for different loadings are given in Chapter XVI.

GENERAL SPECIFICATIONS FOR STEEL FRAME BUILDINGS.*

BY

MILO S. KETCHUM,

M. Am. Soc. C. E.

FOURTH EDITION.

1921.

PART I. DESIGN.

1. **Height of Building.**—The height of the building shall be the distance from the top of the masonry to the under side of the bottom chord of the truss.

2. **Dimensions of Building.**—The width and length of the building shall be the extreme distance out to out of framing or sheathing.

3. **Length of Span.**—The length of trusses and girders in calculating stresses shall be considered as the distance from center to center of end bearings when supported, and from end to end when fastened between columns by connection angles.

4. **Pitch of Roof.**—The pitch of roof for corrugated steel shall preferably be not less than $\frac{1}{4}$ (6 in. in 12 in.), and in no case less than $\frac{1}{5}$. For a pitch less than $\frac{1}{5}$ some other covering than corrugated steel shall be used.

5. **Spacing of Trusses.**—Trusses shall be spaced so that simple shapes may be used for purlins. The spacing should be about 16 ft. for spans of, say, 50 ft. and about 20 to 22 ft. for spans of, say, 100 ft. For longer spans than 100 ft. the purlins may be trussed and the spacing may be increased.

6. **Spacing of Purlins.**—Purlins shall be spaced not to exceed 4 ft. 9 in. where corrugated steel is used, and shall preferably be placed at panel points of the trusses.

7. **Form of Trusses.**—The trusses shall preferably be of the Fink type with panels so subdivided that panel points will come under the purlins. If it is not practicable to place the purlins at panel points, the upper chords of the trusses shall be designed to take both the flexural and direct stresses. Trusses shall preferably be riveted trusses.

Trusses supported on masonry walls shall have one end supported on sliding plates for spans up to 70 ft.; for greater lengths of span, rollers or a rocker shall be used. No rollers with a diameter less than 4 in. shall be used.

8. **Bracing.**—Roof trusses supported on masonry walls or on columns, and transverse bents shall be braced in pairs. The pairs of trusses and transverse bents shall have bracing in the planes of the top and bottom chords, and, unless rigidly braced by other means, shall have transverse bracing between the trusses located approximately at the third points of the lower chord. The pairs of trusses and transverse bents shall be connected by rigid bracing in the plane of the lower chords in line with the lower chords of the transverse bracing. Steel frame buildings without effective knee braces shall have diagonal bracing extending between all pairs of trusses so arranged as to transmit the wind loads to the ends of the building, and the sides and the end bents shall be braced to transmit the wind loads.

Bracing in the plane of the lower chords shall be stiff; bracing in the planes of the top chords, sides and ends may be made adjustable.

9. **Field Connections.**—All field connections of the steel framework shall be riveted, except the connections of purlins and girts, which may be field-bolted.

10. **Proposals.**—Contractors in submitting proposals shall furnish complete stress sheets, general plans of the proposed structures giving sizes of material, and such detail drawings as will clearly show the dimensions of the parts, modes of construction and sectional areas.

* Reprinted from the author's "Steel Mill Buildings," Fourth Edition.

11. **Detail Plans.**—The successful contractor shall furnish all working drawings required by the engineer free of cost. Working drawings shall, as far as possible, be made on standard size sheets 24 in. × 36 in. out to out, 22 in. × 34 in. inside the inner border lines.

12. **Approval of Plans.**—No work shall be commenced nor materials ordered until the working drawings are approved in writing by the engineer. The contractor shall be responsible for dimensions and details on the working plans, and the approval of the detail plans by the engineer shall not relieve the contractor of this responsibility.

PART II. LOADS.

13. The trusses shall be designed to carry the following loads:

14. **DEAD LOADS. Weight of Trusses.**—The weight of trusses per sq. ft. of horizontal projection, up to 150 ft. span, shall be calculated by the formula

$$W = \frac{P}{45}\left(1 + \frac{L}{5\sqrt{A}}\right) \tag{1}$$

where W = weight of trusses per sq. ft. of horizontal projection;
P = capacity of truss in pounds per sq. ft. of horizontal projection;
L = span of the truss in feet;
A = distance between trusses in feet.

15. **Weight of Covering. Corrugated Steel.**—The weight of corrugated steel shall be taken from Table I.

TABLE I.

WEIGHT OF FLAT, AND CORRUGATED STEEL SHEETS WITH 2½-INCH CORRUGATIONS.

Gage No.	Thickness in inches.	Weight per Square (100 sq. ft.)			
		Flat Sheets.		Corrugated Sheets.	
		Black.	Galvanized.	Black Painted.	Galvanized.
16	.0625	250	266	271	286
18	.0500	200	216	217	232
20	.0375	150	166	163	178
22	.0313	125	141	136	151
24	.0250	100	116	110	124
26	.0188	75	91	83	98
28	.0156	63	79	68	85

When two corrugations side lap and six in. end lap are used, add 20 per cent to the above weights; when one corrugation side lap and four in. end lap are used, add 15 per cent to the above weights to obtain weight of corrugated steel laid. For paint add 2 lb. per square. The weight of covering shall be reduced to weight per sq. ft. of horizontal projection before combining with the weight of trusses.

16. **Slate.**—Slate laid with 3 in. lap shall be taken at a weight of 7½ lb. per sq. ft. of inclined roof surface for $\frac{3}{16}$ in. slate 6 in. × 12 in., and 6½ lb. per sq. ft. of inclined roof surface for $\frac{3}{16}$ in. slate 12 in. × 24 in., and proportional for other sizes.

17. **Tile.**—Terra-cotta tile roofing weighs about 6 lb. per sq. ft. for tile 1 in. thick; the actual weight of tile and other roof coverings not named shall be used.

18. **Sheathing and Purlins.**—Sheathing of dry pine lumber shall be assumed to weigh 3 lb. per ft. and dry oak purlins 4 lb. per ft. board measure.

19. **Miscellaneous Loads.**—The exact weight of sheathing, purlins, bracing, ventilators, cranes, etc., shall be calculated.

20. **SNOW LOADS.**—Snow loads shall be assumed as follows:—For a latitude of 40° the snow load in lb. per sq. ft. of horizontal projection shall be; for roofs with ⅙ pitch (18° 15′), 25 lb.; ⅕ pitch (21° 47′), 20 lb.; ¼ pitch (26° 34′), 15 lb.; ⅓ pitch (33° 40′) and over, 10 lb. For a latitude of 50° the snow load in lb. per sq. ft. of horizontal projection shall be; for roofs with ⅙ pitch and less, 50 lb.; ⅕ pitch, 40 lb.; ¼ pitch, 30 lb.; ⅓ pitch, and over 20 lb. Snow loads for other latitudes shall be taken proportional.

For Pacific coast and arid regions, use one-half of the snow loads above specified.

All roofs shall be assumed to carry a minimum snow load or ice load of 10 lb. per sq. ft. of horizontal projection, at the time of maximum wind load.

21. **WIND LOADS.**—The normal wind load on trusses shall be computed by Duchemin's formula

$$P_n = P \frac{2 \sin A}{1 + \sin^2 A} \tag{2}$$

where P_n = normal wind pressure per sq. ft.; A = angle of roof surface with horizontal, and P = 30 lb. per sq. ft.; except for exposed locations where P = 40 lb. per sq. ft. shall be used.

Normal pressures for a horizontal wind pressure of 30 lb. per sq. ft. as calculated by Duchemin's formula are given in Table II.

TABLE II.

NORMAL WIND PRESSURE ON ROOFS FOR HORIZONTAL WIND PRESSURE OF 30 LB. PER SQ. FT. BY DUCHEMIN'S FORMULA, (2).

Angle of Roof with Horizontal. A	Normal Pressure lb. per sq. ft. P_n	Angle of Roof with Horizontal. A	Normal Pressure lb. per sq. ft. P_n
5°	5.1	25°	21.6
10°	10.2	¼ pitch	22.4
15°	14.5	30°	24.0
⅙ pitch	17.2	40°	27.3
20°	18.3	½ pitch	28.3
⅕ pitch	20.0	55° to 90°	30.0

The sides and ends of buildings shall be computed for a normal wind load of 20 lb. per sq. ft. of exposed surface for buildings 30 ft. and less to the eaves; 30 lb. per sq. ft. of exposed surface for buildings 60 ft. to the eaves, and in proportion for intermediate heights.

22. In steel frame buildings having efficient knee-braced bents and also so braced as to transmit wind loads through the planes of the upper and lower chords and sides and ends as in § 8, the wind load may be assumed as taken equally by the two systems of bracing. In which case, the transverse bents may be designed to carry one-half the wind loads specified in § 21.

23. The wind pressure on circular tanks or chimneys shall be taken as 20 lb. per sq. ft. on the vertical projection of the surface.

24. **Mine Buildings.**—Mine, smelter and other buildings exposed to the action of corrosive gases shall have their dead loads increased 25 per cent.

25. **Live Loads.**—Concentrated loads due to cranes, shafting, etc., shall be provided for. In addition to vertical loads due to cranes, the crane girders and the structure shall be designed to withstand a lateral or a transverse loading each equal to twenty per cent. of the lifting capacity of the crane, divided equally between all the wheels of the crane, and applied in the plane of the center of gravity of the top of the flange of the crane girder.

26. **Purlins.**—Purlins shall be designed to carry the actual weight of the covering, roofing and purlins, but shall always be designed for a normal load of not less than 30 lb. per sq. ft., § 57.

27. **Girts.**—Girts shall be designed for a normal load of not less than 20 lb. per sq. ft., § 57.

28. **Roof Covering.**—Roof covering shall be designed for a normal load of not less than 30 lb. per sq. ft.

29. **Minimum Loads.**—No roof shall, however, be designed for an equivalent load of less than 30 lb. per sq. ft. of horizontal projection.

30. **Loads on Foundations.**—The loads on foundations shall not exceed the following in tons per sq. ft.:

Ordinary clay and dry sand mixed with clay	2
Dry sand and dry clay	3
Hard clay and firm coarse sand	4
Firm coarse sand and gravel	5
Shale rock	8
Hard rock	20

For all soils inferior to the above, such as loam, etc., never more than one ton per sq. ft.

31. **Stresses in Masonry.**—The allowable stresses in masonry when used in walls and foundations shall not exceed the following:

	Tons per Sq. Ft.	Lb. per Sq. In.
Common brick, Portland cement mortar	12	170
Hard burned brick, Portland cement mortar	15	210
Rubble masonry, Portland cement mortar	12	170
First class masonry, crystalline sandstone or limestone	25	350
First class masonry, granite	30	420
Portland cement concrete, 1–3–5	20	280
Portland cement concrete, 1–2–4	30	420

32. **Pressures on Masonry.**—The pressure of column bases, beams, etc., on masonry shall not exceed the following in pounds per sq. in.:

Brick work with cement mortar	250
Rubble masonry with cement mortar	250
Portland cement concrete, 1–2–4	600
First class dimension sandstone or limestone	400
First class granite	600

33. **Loads on Timber Piles.**—The maximum load carried by a pile shall not exceed 40,000 lb., or 600 lb. per sq. in. of its average cross-section. The allowable load on piles driven with a drop hammer shall be determined by the formula $P = \frac{2W \cdot h}{s + 1}$. Where P = safe load on pile in tons; W = weight of hammer in tons; h = free fall of hammer in ft.; s = average penetration for the last six blows of the hammer in in. Where a steam hammer is used, $\frac{1}{10}$ is to be used in place of unity in the denominator of the right hand member of the formula.

Piles shall have a penetration of not less than 10 ft. in hard material, such as gravel, and not less than 15 ft. in loam or soft material.

PART III. ALLOWABLE UNIT STRESSES AND PROPORTION OF PARTS.

34. **Allowable Stresses.**—In proportioning the different parts of the structure the maximum stresses due to the combinations of the dead and wind load; dead and snow load; or dead, minimum snow and wind load are to be provided for. Concentrated loads where they occur must be provided for.

35. **Impact.**—For structures carrying cranes and traveling machinery, 25 per cent shall be added to provide for the effect of vibration and impact.

36. **Compressive Stress.**—Allowable Unit Compressive Stress for Structural Steel. For direct dead, snow and wind loads

$$S = 16{,}000 - 70\frac{l}{r}$$

where S = allowable unit stress in lb. per sq. in.;
l = length of member in inches c. to c. of end connections;
r = least radius of gyration of the member in inches.

The maximum value of S shall be 14,000 lb. per sq. in.

37. **Tensile Stress.**—Allowable Unit Tensile Stresses for Structural Steel. For direct dead, snow and wind loads.

	Lb. per Sq. In.
Shapes, main members, net section	16,000
Bars	16,000
Bottom flanges of rolled beams	16,000
Shapes, laterals, net section	20,000
Steel bars for laterals	20,000

38. **Bending.**—Bending; on extreme fibers of rolled shapes, built sections and girders;

net section	16,000
on cast iron	3,000
on extreme fibers of pins	24,000

39. **Shearing.**—

Shearing; shop driven rivets and pins	12,000
field driven rivets and turned bolts	10,000
plate girder webs; net section	10,000
cast iron	1,500

40. Bearing.—Bearing; shop driven rivets and pins 24,000
field driven rivets and turned bolts 20,000
cast iron 12,000
granite masonry and Portland cement concrete 600
sandstone and limestone 400
expansion rollers; per lineal inch 600*d*
cast iron expansion rockers; per lineal inch 300*d*
where "*d*" is the diameter of the roller or rocker in inches.

Rivets shall not be used in direct tension, except for lateral bracing where unavoidable; in which case the value for direct tension on the rivet shall be taken the same as for single shear.

Field bolts, when allowed, shall be spaced for stresses two-thirds those allowed for field rivets.

41. **Maximum Length of Compression Members.**—No compression member shall have a length exceeding 125 times its least radius of gyration for main members, nor 150 times its least radius of gyration for laterals and sub-members. The length of a main tension member in which the stress is reversed by wind shall not exceed 150 times its least radius of gyration.

42. **Maximum Length of Tension Members.**—The length of riveted tension members in horizontal or inclined position shall not exceed 200 times their radius of gyration except for wind bracing, which members may have a length equal to 250 times the least radius of gyration. The distance center to center of end connections of the member is to be considered the effective length.

43. **Alternate Stress.**—Members and connections subject to alternate stresses shall be designed to take each kind of stress.

44. **Combined Stress.**—Members subject to combined direct and bending stresses shall be proportioned according to the following formula:

$$S = \frac{P}{A} + \frac{M \cdot c}{I \pm \dfrac{P \cdot l^2}{10E}}$$

where S = stress in lb. per sq. in. in extreme fiber;
P = direct load in lb.;
A = area of member in sq. in.;
M = bending moment in in.-lb.;
c = distance from neutral axis to extreme fiber in inches;
I = moment of inertia of member;
l = length of member, or distance from point of zero moment to end of member in inches;
E = modulus of elasticity = 30,000,000 lb. per sq. in.

When combined direct and flexural stress due to wind is considered, 50 per cent may be added to the above allowable tensile and compressive stresses.

When the combined stress due to oblique loading of purlins and girts is considered, 25 per cent may be added to allowable stresses.

45. **Stress Due to Weight of Member.**—Where the stress due to the weight of the member or due to an eccentric load exceeds the allowable stress for direct loads by more than 10 per cent, the section shall be increased until the total stress does not exceed the above allowable stress for direct loads by more than 10 per cent.

46. **Angles in Tension.**—When single-angle members subject to direct tension are fastened by one leg, only seventy-five per cent of the net area shall be considered effective. Angles with lug angle connections shall not be considered as fastened by both legs.

47. **Net Section.**—In members subject to tensile stresses full allowance shall be made for reduction of section by rivet-holes, screw-threads, etc. In calculating net area the rivet-holes shall be taken as having a diameter ⅛ in. greater than the normal size of rivet.

The net section of riveted members shall be the least area which can be obtained by deducting from the gross sectional area the areas of holes cut by any plane perpendicular to the axis of the member and parts of the areas of other holes on one side of the plane, within a distance of 4 inches, and which are on other gage lines than those of the holes cut by the plane, the parts being determined by the formula:

$$A(1 - p/4),$$

in which A = the area of the hole, and

p = the distance in inches of the center of the hole from the plane.

48. **Minimum Sections.**—The minimum thickness of plates shall be one-quarter ($\frac{1}{4}$) in., except for fillers. Minimum angles shall be 2 in. by 2 in. by $\frac{1}{4}$ in. The webs of channels shall have a minimum thickness of 0.18 in. The minimum thickness of connection plates of trusses shall be three-eighths ($\frac{3}{8}$) in. The minimum thickness of metal in base plates of columns shall be five-eighths ($\frac{5}{8}$) in. The minimum thickness of metal in head frames, rock houses, coal tipples, washers and breakers shall be five-sixteenths ($\frac{5}{16}$) in. except for fillers. No upset rods, except sag rods, may be less than five-eighths ($\frac{5}{8}$) in. in diameter. Sag rods may be as small as one-half ($\frac{1}{2}$) in. if the ends are properly upset.

49. **Initial Stress.**—Laterals shall be designed for the maximum stresses due to 5,000 pounds initial tension and the maximum stress due to wind.

50. **Design of Plate Girders.**—Plate girders shall be proportioned either by the moment of inertia of their net section; or by assuming that the flanges are concentrated at their centers of gravity, in which case one-eighth of the gross section of the web, if properly spliced, may be used as flange section. The thickness of web plates shall be not less than $\frac{5}{16}$ in., nor less than 1/160 of the unsupported distance between flange angles.

51. **Compression Flanges.**—Compression flanges of plate girders shall have at least the same sectional area as the tension flanges, and shall not have a stress per sq. in. on the gross area greater than $16,000 - 150\ l/b$, where l = unsupported distance, and b = width of flange, both in inches. Compression flanges of plate girders shall be stayed transversely when their length is more than thirty times their width.

52. **Web Stiffeners.**—There shall be web stiffeners, generally in pairs, over bearings, at points of concentrated loading, and at other points where the thickness of the web is less than $\frac{1}{60}$ of the unsupported distance between flange angles. The distance between stiffeners shall not exceed that given by the following formula, with a maximum limit of six feet (and not greater than the clear depth of the web): $d = t\ (12,000 - s)/40$. Where d = clear distance between stiffeners of flange angles; t = thickness of web; s = shear in lb. per sq. in.

The stiffeners at ends and at points of concentrated loads shall be proportioned by the formula of paragraph 36, the effective length being assumed as one-half the depth of girder. End stiffeners and those under concentrated loads shall be on fillers and have their outstanding legs as wide as the flange angles will allow and shall fit tightly against them. Intermediate stiffeners may be offset or on fillers, and their outstanding legs shall be not less than one-thirtieth of the depth of girder, plus 2 in.

53. **Flange Rivets.**—The flanges of plate girders shall be connected to the web with a sufficient number of rivets to transfer the total shear at any point in a distance equal to the effective depth of the girder at that point combined with any load that is applied directly on the flange. The wheel loads of crane girders shall be assumed to be distributed over 25 inches. The coefficient of friction of crane girder wheels on steel rails shall be taken as 0.20.

54. **Rolled Beams.**—Rolled beams shall be proportioned by their moment of inertia. The depth of rolled beams in floors shall not be less than one-twentieth ($\frac{1}{20}$) of the span. Where rolled beams or channels are used as roof purlins the depths shall not be less than one-fortieth ($\frac{1}{40}$) of the span. When the unsupported length of rolled beams when used as girders exceeds 20 times the width of flange, b, the unit stress in the flange shall not exceed $16,000 - 150\ l/b$ lb.

55. **Timber.**—The allowable stresses in timber purlins and other timber shall be taken from Table III.

TABLE III.

ALLOWABLE WORKING UNIT STRESSES IN TIMBER, IN POUNDS PER SQUARE INCH.

Kind of Timber.	Transverse Loading, S.	End Bearing, C.	Columns Under 12 Diameters	Bearing Across Fiber.	Shear. Parallel to Grain.	Shear. Longitudinal Shear in Beams.	Modulus of Elasticity, E.
White Oak	1,500	1,500	1,200	600	260	140	1,200,000
Long Leaf Yellow Pine	1,500	1,500	1,200	350	220	150	1,500,000
White Pine and Spruce	1,200	1,200	960	200	125	90	1,200,000
Western Hemlock	1,400	1,500	1,200	300	200	125	1,500,000
Douglas Fir	1,500	1,500	1,200	400	210	140	1,200,000

Columns may be used with a length not exceeding 40 times the least dimension. The unit

stress for lengths of more than 12 times the least dimension shall be reduced by the following formula:

$$P = C - \frac{C}{60}\frac{l}{d}$$

where C = unit stress, as given above for end bearing;
P = allowable unit stress in lb. per sq. in.;
l = length of column in inches;
d = least side of column in inches.

PART IV. COVERING AND FLOORS.

56. **Corrugated Steel.**—Corrugated steel shall generally have 2½ in. corrugations when used for roof and sides of buildings, and 1¼ in. corrugations when used for lining buildings. The minimum gage of corrugated steel shall be No. 22 for roofs, No. 24 for sides, and No. 26 for lining.

The gage of corrugated steel in U. S. standard gage and weight per sq. ft. shall be shown on the general plan.

57. **Spacing of Purlins and Girts.**—The spacing, or center to center distance of purlins carrying a corrugated steel roof without sheathing, shall not exceed the distances given in Table IV for a safe load of 30 lb. per sq. ft. Girts for corrugated steel shall be spaced for a safe load of 20 lb. per sq. ft. as given in Table IV. Corrugated steel sheets shall preferably span two purlin or girt spaces. When sag rods are provided as in § 58 and § 59, purlins and girts shall be designed to carry the normal loads with a maximum unit stress of 16,000 lb. per sq. in.

TABLE IV.

MAXIMUM SPACING FOR PURLINS AND GIRTS SUPPORTING CORRUGATED STEEL.

Gage of Steel, No.	Spacing of Purlins and Girts.	
	Purlins, 30 lb. per sq ft.	Girts, 20 lb. per sq. ft.
16	5 ft. 8 in.	6 ft. 9 in.
18	5 ft. 2 in.	6 ft. 2 in.
20	4 ft. 6 in.	5 ft. 4 in.
22	4 ft. 2 in.	5 ft. 0 in.
24	3 ft. 8 in.	4 ft. 6 in.

58. **Sag Rods.**—With a steel corrugated roof one sag rod, at the center, shall be used for purlin spans of 20 ft. or less, and two sag rods, spaced at the third points, for purlin spans of more than 20 ft. With clay tile, cement tile, slate, gypsum, or similar roofs, one sag rod shall be used for purlin spans of 14 ft. or less, and two sag rods spaced at the third points, for spans of more than 14 ft. Where one sag rod is used, the sag rod on each side of the roof in any panel shall be rigidly connected through the ridge purlins. Where two sag rods are used in any panel, each sag rod shall be rigidly connected with the peak of the nearest truss by means of a diagonal sag rod in the upper purlin space. Sag rods need not be used in roofs having a slope of 3 in. in 12 in., or less. With corrugated steel siding, one sag rod shall be used for all girt spacings of 20 ft. or less, and two sag rods spaced at third points for girt spacings of more than 20 ft.

59. Sag rods shall be designed to carry the component of the dead load of the purlins and roof covering and the maximum snow load parallel to the roof surface, with a unit stress of 16,000 lb. per sq. in. on net section. Sag rods for the sides shall be designed to carry the weight of the side framing and covering with the same allowable unit stresses as for sag rods for purlins. If sag rods are not upset the net section shall be taken as the section having a diameter ⅛ in. less than the diameter of the root of the thread. The minimum size of sag rods shall have a diameter of ½ in. if the ends are upset, or ⅝ in. if the ends are not upset.

60. **End and Side Laps.**—Corrugated steel shall be laid with two corrugations side lap and six inches end lap when used for roofing, and one corrugation side lap and four inches end lap when used for siding.

61. **Fastening.**—Corrugated steel shall be fastened to the purlins and girts by means of galvanized iron straps ¾ in. wide by No. 18 gage, spaced 8 to 12 in. apart; by clinch nails spaced 8 to 12 in. apart; or by nailing directly to spiking strips with 8d barbed nails, spaced 8 in. apart.

Spiking strips shall preferably be used with anti-condensation lining. Bolts, nails and rivets shall always pass through the top of corrugations. Side laps shall be riveted with copper or galvanized iron rivets 8 to 12 in. apart on the roof and 1½ to 2 ft. apart on the sides.

62. **Corrugated Steel Lining.**—Corrugated steel lining on the sides shall be laid with one corrugation side lap and four in. end lap. Girts for corrugated steel lining shall be spaced for a safe load of 20 lb. per sq. ft. as given in Table IV.

63. **Anti-Condensation Lining.**—Anti-condensation roof lining shall be used to prevent dripping in engine houses and similar buildings, and shall be constructed as follows:—(1) Lay wire netting, No. 19, 2-in. mesh, transversely to the purlins, with edges 1½ in. apart, so that when laced together with No. 20 brass wire the netting will be stretched smooth and tight.

(2) On the top of the netting lay asbestos paper weighing 30 lb. to the square of 100 sq. ft., allowing 3 in. for laps. For important work lay one or two thicknesses of building paper on top of the asbestos.

(3) Lay the corrugated steel and fasten to purlins in the usual manner.

If wood purlins are used the wire netting may be fastened to the nailing strip swith ¾ in. staples. Where the purlins are more than 2 ft. 6 in. centers place a line of $\frac{3}{16}$ in. bolts between purlins, about 2 ft. centers, with washers 1 in. × 4 in. × ⅛ in. to prevent netting from sagging.

64. **Flashing.**—Valleys or corners around stacks shall have flashing extending at least 12 in. above where water will stand, and shall be riveted or soldered, if necessary, to prevent leakage.

Flashing shall be provided above doors and windows. Flashing shall be made of steel not lighter than No. 20 gage.

65. **Ridge Roll.**—All ridges shall have a ridge roll, the same thickness as the corrugated steel, securely fastened to the corrugated steel.

66. **Corner Finish.**—All corners shall be covered with standard corner finish, the same thickness as the corrugated steel, securely fastened to the corrugated steel.

67. **Cornice.**—At the gable ends the corrugated steel on the roof shall be securely fastened to a finish angle or channel connected to the end of the purlins, or, where molded cornices are used, to a piece of timber fastened to the ends of the purlins. Cornice shall be made of steel not lighter than No. 20 gage.

68. **Gutters and Conductors.**—Gutters and conductors shall be furnished at least equal to the requirements of the following table:

Span of Roof.	Gutter.	Conductor.
Up to 50 ft.	6 in.	4 in. every 40 ft.
50 ft. to 70 ft.	7 in.	5 in. every 40 ft.
70 ft. to 100 ft.	8 in.	5 in. every 40 ft.

Gutters shall have a slope of at least 1 in. in 15 ft. Gutters and conductors shall be made of galvanized steel not lighter than No. 20 gage.

69. **Ventilators.**—Ventilators shall be provided and located so as to properly ventilate the building. They shall have a net opening for each 100 sq. ft. of floor space as follows: not less than one-fourth sq. ft. for clean machine shops and similar buildings; not less than one sq. ft. for dirty machine shops; not less than four sq. ft. for mills; and not less than six sq. ft. for forge shops, foundries and smelters.

70. **Shutters and Louvres.**—Openings in ventilators shall be provided with shutters, sash, or louvres, or may be left open as specified.

Shutters must be provided with a satisfactory device for opening and closing.

Louvres must be designed to prevent the blowing in of rain and snow, and must be made stiff so that no appreciable sagging will occur. They shall be made of not less than No. 20 gage galvanized steel for flat louvres, and No. 24 gage galvanized steel for corrugated louvres.

71. **Circular Ventilators.**—Circular ventilators, when used, must be designed so as to prevent down drafts. Net opening only shall be used in calculations.

72. **Windows and Skylights.**—Where buildings are lighted by windows the clear window area shall not be less than 20 per cent of the floor area, nor less than 10 per cent of the area of the entire exterior surface in mill buildings, nor less than 20 per cent of the area of the entire exterior surface in machine shops, factories and other buildings in which men are required to work at machines. Skylights shall be used where the required window area cannot be provided in the sides and ends of buildings.

Where buildings are lighted by windows having the sills not more than 4 ft. above the floor, the span of the building shall not exceed 2 times the height of the top of the windows where buildings are lighted by windows in one side, or 4 times the height of the top of the windows where buildings are lighted by windows in both sides. Where the span of the building is greater than is permitted

by the preceding requirement, the necessary illumination shall be provided either by prism glass in side walls or by skylights. Skylights shall have such an area and shall be so arranged that light coming through the skylight making an angle of not more than 45° with the vertical shall cover the entire horizontal area at a distance of 6 feet above the floor; or the light may be diffused by means of ribbed glass or prisms or by reflection from the ceiling to obtain equally satisfactory illumination. In saw tooth roofs the inner surface of the roof shall be light colored or shall be painted with a paint that will reflect the light and make the illumination uniform and effective. All windows or skylights admitting direct sunlight shall be provided with muslin or other satisfactory shades.

73. **Skylights.**—Skylights shall be glazed with wire glass, or wire netting shall be stretched beneath the skylights to prevent the broken glass from falling into the building. Where there is danger of the skylight glass being broken by objects falling on it, a wire netting guard shall be provided on the outside.

Skylight glass shall be carefully set, special care being used to prevent leakage. Leakage and condensation on the inner surface of the glass shall be carried to the down-spouts, or outside the building by condensation gutters.

74. **Wood Sash.**—Window glass set in wood sash up to 12 in. × 14 in. may be single strength, over 12 in. × 14 in. the glass shall be double strength. Window glass shall be A grade except in smelters, foundries, forge shops and similar structures, where it may be B grade. The sash and frames shall be constructed of white pine. Where buildings are exposed to fire hazard the windows shall have wire glass set in metal sash and frames.

Windows with wood sash in sides of buildings shall be made with counter-balanced sash, and in ventilators shall be made with sliding or swing sash. All swinging windows shall be provided with a satisfactory operating device.

75. **Wire Glass.**—Wire glass shall have a thickness of not less than $\frac{1}{4}$ in. The wire mesh shall be not larger than $\frac{7}{8}$ in., and the thickness of the wire shall not be less than No. 24 B. & S. gage for single wire, or No. 27 B. & S. gage for twisted double wire. The wire shall be practically midway between the two surfaces of glass. Lights shall not have a greater area than 720 sq. in., or more than 54 in. vertical and 48 in. horizontal. Lights of glass shall preferably be 12 in. by 18 in. or 14 in. by 20 in. The selvage shall be removed from the glass before setting. The bearing of glass in grooves shall not be less than $\frac{3}{8}$ in. at all points, and there shall be a clearance of $\frac{1}{8}$ in. between the edge of the glass and the frame.

76. **Steel Sash.**—Steel sash shall be made with solid sections. The maximum size of steel sash shall be 100 sq. ft. where no ventilators are used, and 70 sq. ft. where ventilators occupy two-thirds of the window area and proportional for intermediate amount of ventilators. Steel sash shall be glazed with special glazing clips and with glazing putty. All sash shall be provided with locking devices, and other hardware as specified.

77. **Doors.**—Doors are to be furnished as specified and are to be provided with hinges, tracks, locks, and bolts. Single doors up to 4 ft. and double doors up to 8 ft. shall preferably be swung on hinges; large doors, double and single, shall be arranged to slide on overhead tracks, or may be counterbalanced to lift up between vertical guides.

Steel doors shall be firmly braced. Unless otherwise specified, steel doors shall be covered with No. 24 corrugated steel with $1\frac{1}{4}$ in. corrugations.

The frames of sandwich doors shall be made of two layers of $\frac{7}{8}$ in. matched white pine, placed diagonally and firmly nailed with clinch nails. The frame shall be covered on each side with a layer of No. 24 corrugated steel with $1\frac{1}{4}$ in. corrugations. Locks and all other necessary hardware shall be furnished for all windows and doors.

78. **TAR AND GRAVEL ROOF.**—Tar and gravel roofs are called three-, four-, five-ply, etc., depending upon the number of layers of roofing felt. Tar and gravel roofs may be laid upon timber sheathing or upon concrete or gypsum slabs.

79. **Specifications for Five-Ply Tar and Gravel Roof on Board Sheathing.**—The materials used in making the roof are one (1) thickness of sheathing paper or unsaturated felt, five (5) thicknesses of saturated felt weighing not less than fifteen (15) pounds per square of one hundred (100) square feet, single thickness, and not less than one hundred and fifty (150) pounds of pitch, and not less than four hundred (400) pounds of gravel or three hundred (300) pounds of slag from $\frac{1}{4}$ to $\frac{5}{8}$ in. in size, free from dirt, per square of one hundred (100) square feet of completed roof.

80. The material shall be applied as follows: First, lay the sheathing or unsaturated felt, lapping each sheet one inch over the preceding one. Second, lay two (2) thicknesses of tarred felt, lapping each sheet seventeen (17) inches over the preceding one, nailing as often as may be necessary to hold the sheets in place until the remaining felt is applied. Third, coat the entire surface of this two-ply layer with hot pitch, mopped on uniformly. Fourth, apply three (3) thicknesses of felt, lapping each sheet twenty-two (22) inches over the preceding one, mopping

with hot pitch the full width of the 22 inches between the plies, so that in no case shall felt touch felt. Such nailing as is necessary shall be done so that all nails will be covered by not less than two plies of felt. Fifth, spread over the entire surface of the roof a uniform coating of pitch, into which, while hot, imbed the gravel or slag. The gravel or slag in all cases must be dry.

81. **Specifications for Five-Ply Tar and Gravel Roof on Concrete Sheathing.**—The materials used shall be the same as for tar and gravel roof on timber sheating, except that the one thickness of sheathing paper or unsaturated felt may be omitted.

82. The materials shall be applied as follows: First, coat the concrete with hot pitch, mopped on uniformly. Second, lay two (2) thicknesses of tarred felt, lapping each sheet seventeen (17) inches over the preceding one, and mop with hot pitch the full width of the 17-inch lap, so that in no case shall felt touch felt. Third, coat the entire surface with hot pitch, mopped on uniformly. Fourth, lay three (3) thicknesses of felt, lapping each sheet twenty-two (22) inches over the preceding one, mopping with hot pitch the full width of the 22-inch lap between the plies, so that in no case shall felt touch felt. Fifth, spread the entire surface of the roof with a uniform coat of pitch, into which, while hot, imbed gravel or slag.

Tar and gravel roof shall be laid on gypsum sheathing in the same manner as on concrete sheathing.

83. **SPECIFICATIONS FOR CEMENT FLOOR ON A CONCRETE BASE. Materials.**—The cement used shall be first-class Portland cement, and shall pass the standards of the American Society for Testing Materials. The sand for the top finish shall be clean and sharp and shall be retained on a No. 30 sieve and shall have passed the No. 20 sieve. Broken stone for the top finish shall pass a ½ in. screen and shall be retained on the No. 20 screen. Dust shall be excluded. The sand for the base shall be clean and sharp. The aggregate for the base shall be of broken stone or gravel and shall pass a 2 in. ring.

84. **Base.**—On a thoroughly tamped and compacted subgrade the concrete for the base shall be laid and thoroughly tamped. The base shall not be less than 2½ in. thick. Concrete for the base shall be thoroughly mixed with sufficient water so that some tamping is required to bring the moisture to the surface. If old concrete is used for the base the surface shall be roughened and thoroughly cleaned so that the new mortar will adhere. The roughened surface of old concrete shall then be thoroughly wet so that the base will not draw water from the finish when the latter is applied. Before scrubbing the base with grout the excess water shall be removed.

85. **Finish.**—With old concrete the surface of the base shall first be scrubbed with a thin grout of pure cement, rubbed in with a broom. On top of this, before the thin coat is set, a coat of finish mixed in the proportions of one part Portland cement, one part stone broken to pass a ½ in. ring, and one part sand shall be troweled on, using as much pressure as possible, so that it will take a firm bond. After the finish has been applied to the desired thickness, preferably 2 in., it should be screeded and floated to a true surface. Between the time of initial and final set it shall be finished by skilled workmen with steel trowels and shall be worked to final surface. Under no condition shall a dryer be used, nor shall water be added to make the material work easily.

86. **SPECIFICATIONS FOR WOOD FLOOR ON A TAR CONCRETE BASE. Floor Sleepers.**—Sleepers for carrying the timber floor shall be 3 in. × 3 in. placed 18 in. c. to c. After the subgrade has been thoroughly tamped and rolled to an elevation of 4½ in. below the tops of the sleepers, the sleepers shall be placed in position and supported on stakes driven in the subgrade. Before depositing the tar concrete the sleepers must be brought to a true level.

87. **Tar Concrete Base.**—The tar concrete base shall be not less than 4½ in. thick and shall be laid as follows: First, a layer three (3) inches thick of coarse, screened gravel thoroughly mixed with tar, and tamped to a hard level surface. Second, on this bed spread a top dressing 1½ inches thick of sand heated and thoroughly mixed with coal tar pitch, in the proportions of one (1) part pitch to three (3) parts tar. The gravel, sand and tar shall be heated to from 200 to 300 degrees F., and shall be thoroughly mixed and carefully tamped into place.

88. **Plank Sub-Floor.**—The floor plank shall be of sound hemlock or pine not less than 2 inches thick, planed on one side and one edge to an even thickness and width. The floor plank is to be toe-nailed with 4 in. wire nails.

89. **Finished Flooring.**—The finished flooring is to be of maple of clear stock, ⅞-in. finished thickness, thoroughly air and kiln dried and not over 4 inches wide. The floor is to be planed to an even thickness, the edges jointed, and the underside channeled or ploughed. The finished floor is to be laid at right angles to the sub-floor, and each board neatly fitted at the ends, breaking joints at random. The floor is to be final nailed with 10 d. or 3-in. wire nails, nailed in diagonal rows 16 inches apart across the boards, with two (2) nails in each row in every board. The floor to be finished off perfectly smooth on completion.

90. The finished flooring is not to be taken into the building or laid until the tar concrete base and sub-floor plank are thoroughly dried.

PART V. DETAILS OF CONSTRUCTION.

91. **Details.**—All connections and details shall be of sufficient strength to develop the full strength of the member.

92. **Pitch of Rivets.**—The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3 in. for ⅞-in. rivets, 2½ in. for ¾-in. rivets, and 2 in. for ⅝-in. rivets. The maximum pitch in the lines of stress for members composed of plates and shapes shall be 16 times the thickness of the thinnest outside plate or 6 in. For angles with two gage lines and rivets staggered, the maximum shall be twice the above in each line. Where two or more plates are used in contact, rivets not more than 12 in. apart in either direction shall be used to hold the plates well together.

93. **Edge Distance.**—The minimum distance from the center of any rivet hole to a sheared edge shall be 1½ in. for ⅞-in. rivets, 1¼ in. for ¾-in. rivets, and 1⅛ in. for ⅝-in. rivets, and to a rolled edge 1¼, 1⅛ and 1 in., respectively. The maximum distance from any edge shall be eight times the thickness of the plate, but shall not exceed 6 in.

94. **Maximum Diameter.**—The diameter of the rivets in any angle carrying calculated stress shall not exceed one-quarter the width of the leg in which they are driven. In minor parts ⅞-in. rivets may be used in 3-in. angles, ¾-in. rivets in 2½-in. angles, and ⅝-in. rivets in 2-in. angles.

95. **Long Rivets.**—Rivets carrying calculated stress and whose grip exceeds four diameters shall be increased in number at least one per cent for each additional $\frac{1}{16}$ in. of grip.

96. **Pitch at Ends.**—The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets, for a length equal to one and one-half times the maximum width of member.

97. **Diameter of Punch and Die.**—The diameter of the punch and die shall be as specified in § 157.

98. **Connections.**—All connections shall be of sufficient strength to develop the full strength of the member. No connections except for lacing bars shall have less than two rivets. All field connections except lacing bars shall have not less than three rivets.

99. **Flange Plates.**—The flange plates of all girders shall not extend beyond the outer line of rivets connecting them to the angles more than 6 in. nor more than eight times the thickness of the thinnest plate.

100. **Web Stiffeners.**—Web stiffeners shall be in pairs, and shall have a close fit against flange angles. The stiffeners at the ends of plate girders shall have filler plates. Intermediate stiffeners may have fillers or be crimped over the flange angles. The rivet pitch in stiffeners shall not be greater than 5 in.

101. **Web Splices.**—Web plates shall be spliced at all points by a plate on each side of the web, capable of transmitting the shearing and bending stresses through the splice rivets.

102. **Riveted Tension Members.**—Pin connected riveted tension members shall have a net section through the pin hole 25 per cent in excess of the required net section of the member. The net section back of the pin hole in line of the center of the pin shall be at least 0.75 of the net section through the pin hole.

103. **Upset Rods.**—All rods with screw ends, except sag rods, must be upset at the ends so that the diameter at the base of the threads shall be $\frac{1}{16}$ inch larger than any part of the body of the bar.

104. **Upper Chords.**—Upper chords of trusses shall have symmetrical cross-sections, and shall preferably consist of two angles back to back.

105. **Compression Members.**—All other compression members for roof trusses, except sub-struts, shall be composed of sections symmetrically placed. Sub-struts may consist of a single section.

106. **Columns.**—Side posts which take flexure shall preferably be composed of 4 angles and a plate. In calculating the least moment of inertia of columns made of 4 angles and a web plate, the web plate may be omitted. Where side posts do not take flexure and carry heavy loads they shall preferably be composed of two channels laced, or of two channels with a center diaphragm.

107. Posts in end framing shall preferably be composed of I-beams or 4 angles laced. Corner columns shall preferably be composed of one angle.

108. **Crane Posts.**—The cross-bending stress due to eccentric loading in columns carrying cranes shall be calculated. Crane girders carrying heavy cranes shall be carried on independent columns.

109. **Batten Plates.**—The open sides of all compression members shall be stayed by batten plates at the ends and diagonal lattice-work at intermediate points. The batten plates must be placed as near the ends as practicable, and shall have a length not less than the greatest width of the member or 1½ times its least width.

110. **Lacing Bars.**—The lacing of compression members shall be proportioned to resist a shearing stress of 2½ per cent of the direct stress. The minimum width of lacing bars shall be 1¾ in. for members 6 in. in width, 2 in. for members 9 in. in width, 2¼ in. for members 12 in. in width, 2½ in. for members 15 in. in width, or 3 in. for members 18 in. and over in width. Single lacing bars shall have a thickness not less than one-fortieth, or double lacing bars connected by a rivet at the intersection, not less than one-sixtieth of the distance between the rivets connecting them to the members. They shall be inclined at an angle not less than 60° to the axis of the member for single lacing, nor less than 45° for double lacing with riveted intersections. Lacing bars shall be so spaced that the portion of the flange included between their connection shall be as strong as the member as a whole. The pitch of the lacing bars must not exceed the width of the channel plus nine inches.

111. **Pin Plates.**—All pin holes shall be reinforced by additional material when necessary, so as not to exceed the allowable pressure on the pins. These reinforcing plates must contain enough rivets to transfer the proportion of pressure which comes upon them, and at least one plate on each side shall extend not less than 6 in. beyond the edge of the batten plate.

112. **Splices.**—In compression members joints with abutting faces planed shall be placed as near the panel points as possible, and must be spliced on all sides with at least two rows of rivets on each side of the joint. Joints with abutting faces not planed must be fully spliced.

113. **Splices.**—Joints in tension members shall be fully spliced.

114. **Tension Members.**—Tension members shall preferably be composed of angles or shapes capable of taking compression as well as tension. Flats riveted at the ends shall not be used.

115. Main tension members shall preferably be made of 2 angles, 2 angles and a plate, or 2 channels laced. Secondary tension members may be made of a single shape.

116. **Eye-Bars.**—Heads of eye-bars shall be so proportioned as to develop the full strength of the bar. The heads shall be forged and not welded.

117. **Pins.**—Pins must be turned true to size and straight, and must be driven to place by means of pilot nuts.

The diameter of pin shall not be less than ¾ of the depth of the widest bar attached to it.

The several members attached to a pin shall be packed so as to produce the least bending moment on the pin, and all vacant spaces must be filled with steel or cast iron fillers.

118. **Bars or Rods.**—Long laterals may be made of bars with clevis or sleeve nut adjustment. Bent loops shall not be used.

119. **Spacing Trusses.**—Trusses shall preferably be spaced so as to allow the use of single pieces of rolled sections for purlins. Trussed purlins shall be avoided if possible.

120. **Purlins and Girts.**—Purlins and girts shall preferably be composed of single sections—channels, angles or Z-bars, placed with web at right angles to the trusses and posts and legs turned down.

121. **Fastening.**—Purlins and girts shall be attached to the top chord of trusses and to columns by means of angle clips with two rivets or two bolts in each leg.

122. **Spacing.**—Purlins for corrugated steel without sheathing shall be spaced at distances apart not to exceed the span as given for a safe load of 30 lb., and girts for a safe load of 20 lb. as given in Table IV.

123. **Timber Purlins.**—Timber purlins and girts shall be attached and spaced the same as steel purlins.

124. **Base Plates.**—Base plates shall never be less than ⅝ in. in thickness, and shall be of sufficient thickness and size so that the pressure on the masonry shall not exceed the allowable pressures in § 32.

125. **Cast Rockers.**—The details of cast iron rockers shall be subject to the special approval of the engineer. The vertical webs of cast iron rockers and pedestals shall be designed for an allowable unit stress of $9{,}000 - 40\, l/r$, where l = height and r = radius of gyration of vertical web, both in inches.

126. **Anchors.**—Columns shall be anchored to the foundations by means of two anchor bolts not less than 1 in. in diameter upset, placed as wide apart as practicable in the plane of the wind. The anchorage shall be calculated to resist one and one-half times the bending moment at the base of the columns.

127. **Lateral Bracing.**—Lateral bracing shall be provided in the plane of the top and bottom chords, sides and ends; knee braces in the transverse bents; and sway bracing wherever necessary, see § 8. Lateral bracing shall be designed for an initial stress of 5,000 lb. in each member, and provision must be made for putting this initial stress into the members in erecting.

128. **Temperature.**—No special provision shall be made for changes in temperature in the length and width of a building with a steel frame, except in glazed roofs where expansion joints shall be provided by bolting joints about every 30 ft. Where trusses rest on masonry walls slotted holes shall be provided in the end bearing plates, and in the purlins and roof covering to provide for a variation in temperature of 150° F. In crane runways or similar structures changes in length due to a variation in temperature of 150° F. shall be provided for either by means of slotted holes, or in calculating the stresses.

PART VI. MATERIALS.

129. **Process of Manufacture.**—Structural steel shall be made by the open-hearth process.

130. **Chemical Composition.**—The steel shall conform to the following requirements as to chemical composition:

	Structural Steel.	Rivet Steel.
Phosphorus..................	not over 0.06 per cent	not over 0.06 per cent
Sulfur.....................	......	" " 0.045 "

131. **Ladle Analyses.**—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus and sulfur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in § 130.

132. **Check Analyses.**—Analyses may be made by the purchaser from finished material representing each melt. The phosphorus and sulfur content thus determined shall not exceed that specified in § 130 by more than 25 per cent.

133. **Tension Tests.**—(*a*) The material shall conform to the following requirements as to tensile properties:

Properties Considered.	Structural Steel.	Rivet Steel.
Tensile strength, lb. per sq. in.................	55,000–65,000	46,000–56,000
Yield point, min., lb. per sq. in................	0.5 tens. str.	0.5 tens. str.
Elongation in 8 in., min., per cent..............	$\frac{1,400,000^*}{\text{Tens. str.}}$	$\frac{1,400,000}{\text{Tens. str.}}$
Elongation in 2 in., min., per cent.......... ...	22	....

* See § 134.

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

134. **Modifications in Elongation.**—(*a*) For structural steel over ¾ in. in thickness, a deduction of 1 from the percentage of elongation in 8 in. specified in § 133(*a*) shall be made for each increase of ⅛ in. in thickness above ¾ in., to a minimum of 18 per cent.

(*b*) For structural steel under $\frac{5}{16}$ in. in thickness, a deduction of 2.5 from the percentage of elongation in 8 in. specified in § 133(*a*) shall be made for each decrease of $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in.

135. **Bend Tests.**—(*a*) The test specimen for plates, shapes and bars, except as specified in paragraphs (*b*) and (*c*), shall bend cold through 180 deg. without cracking on the outside of the bent portion, as follows: For material ¾ in. or under in thickness, flat on itself; for material over ¾ in. to and including 1¼ in. in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material over 1¼ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*) The test specimen for pins, rollers and other bars, when prepared as specified in § 136(*e*), shall bend cold through 180 deg. around a 1-in. pin without cracking on the outside of the bent portion.

(*c*) The test specimen for rivet steel shall bend cold through 180 deg. flat on itself without cracking on the outside of the bent portion.

(**Note.**—These Specifications for structural steel conform with Specifications for Structural Steel for Buildings adopted by American Society for Testing Materials, except that Bessemer steel is not permitted.)

136. Test Specimens.—(*a*) Tension and bend test specimens shall be taken from rolled steel in the condition in which it comes from the rolls, except as specified in paragraph (*b*).

(*b*) Tension and bend test specimens for pins and rollers shall be taken from the finished bars, after annealing when annealing is specified.

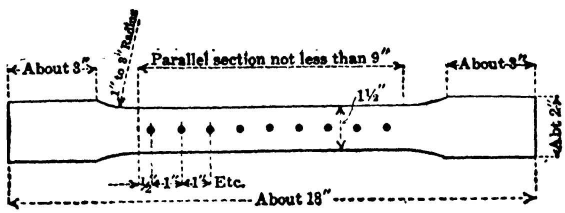


FIG. 1.

(*c*) Tension and bend test specimens for plates, shapes and bars, except as specified in paragraphs (*d*), (*e*) and (*f*), shall be of the full thickness of material as rolled; and may be machined to the form and dimensions shown in Fig. 1, or with both edges parallel.

(*d*) Tension and bend test specimens for plates over 1½ in. in thickness may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.

(*e*) Tension test specimens for pins, rollers and bars over 1½ in. in thickness or diameter may conform to the dimensions shown in Fig. 2. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend test specimens may be 1 by ½ in. in section. The axis of the specimen shall be located at any point midway between the center and surface and shall be parallel to the axis of the bar.

(*f*) Tension and bend test specimens for rivet steel shall be of the full-size section of bars as rolled.

137. Number of Tests.—(*a*) One tension and one bend test shall be made from each melt; except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

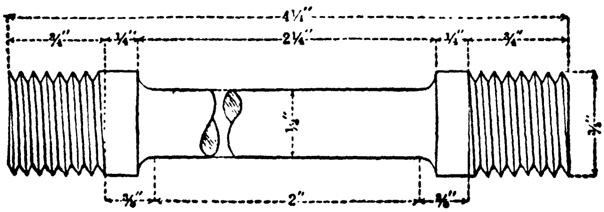


FIG. 2.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 133(*a*) and any part of the fracture is more than ¾ in. from the center of the gage length of a 2-in. specimen or is outside the middle third of the gage length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

138. Permissible Variations.—The cross-section or weight of each piece of steel shall not vary more than 2.5 per cent from that specified; except in the case of sheared plates, which shall be covered by the following permissible variations. One cubic inch of rolled steel is assumed to weigh 0.2833 lb.

(*a*) *When Ordered to Weight per Square Foot:* The weight of each lot[1] in each shipment shall not vary from the weight ordered more than the amount given in Table V.

[1] The term "lot" applied to Table V means all of the plates of each group width and group weight.

TABLE V.

Permissible Variations of Plates Ordered to Weight.

Ordered Weight, Lb. per Sq. Ft.	Permissible Variations in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Ordered Weights.																		Ordered Weight, Lb. per Sq. Ft.
	Under 48 in.		48 to 60 in., excl.		60 to 72 in., excl.		72 to 84 in., excl.		84 to 96 in., excl.		96 to 108 in., excl.		108 to 120 in., excl.		120 to 132 in., excl.		132 in. or over.		
Under 5....	5	3	5.5	3	6	3	7	3	...	...	...	...	...	...	...	...	...	...	Under 5
5 to 7.5 exclusive....	4.5	3	5	3	5.5	3	6	3	...	...	...	...	...	...	...	...	...	...	5 to 7.5 exclusive
7.5 to 10 exclusive....	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	...	...	...	...	7.5 to 10 exclusive
10 to 12.5 exclusive....	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	9	3	10 to 12.5 exclusive
12.5 to 15 exclusive....	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	12.5 to 15 exclusive
15 to 17.5 exclusive....	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	15 to 17.5 exclusive
17.5 to 20 exclusive....	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	17.5 to 20 exclusive
20 to 25 exclusive....	2	2	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	20 to 25 exclusive
25 to 30 exclusive....	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	5	3	25 to 30 exclusive
30 to 40 exclusive....	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	30 to 40 exclusive
40 or over...	2	2	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	40 or over

Note.—The weight per square foot of individual plates shall not vary from the ordered weight by more than 1¼ times the amount given in this table.

(b) *When Ordered to Thickness:* The thickness of each plate shall not vary more than 0.01 in. under that ordered.

The overweight of each lot [2] in each shipment shall not exceed the amount given in Table VI.

[2] The term "lot" applied to Table VI means all of the plates of each group width and group thickness.

TABLE VI.

Permissible Overweights of Plates Ordered to Thickness.

Ordered Thickness, in.	Permissible Excess in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Nominal Weights.									Ordered Thickness, in.
	Under 48 in.	48 to 60 in., excl.	60 to 72 in., excl.	72 to 84 in., excl	84 to 96 in., excl	96 to 108 in., excl	108 to 120 in., excl.	120 to 132 in., excl.	132 in. or over.	
Under ⅛	9	10	12	14	..	..	..	..	..	Under ⅛
⅛ to 3/16 excl.	8	9	10	12	..	..	..	..	..	⅛ to 3/16 excl.
3/16 " ¼ "	7	8	9	10	12	..	..	..	..	3/16 " ¼ "
¼ " 5/16 "	6	7	8	9	10	12	14	16	19	¼ " 5/16 "
5/16 " ⅜ "	5	6	7	8	9	10	12	14	17	5/16 " ⅜ "
⅜ " 7/16 "	4.5	5	6	7	8	9	10	12	15	⅜ " 7/16 "
7/16 " ½ "	4	4.5	5	6	7	8	9	10	13	7/16 " ½ "
½ " ⅝ "	3.5	4	4.5	5	6	7	8	9	11	½ " ⅝ "
⅝ " ¾ "	3	3.5	4	4.5	5	6	7	8	9	⅝ " ¾ "
¾ " 1 "	2.5	3	3.5	4	4.5	5	6	7	8	¾ " 1 "
1 or over	2.5	2.5	3	3.5	4	4.5	5	6	7	1 or over

139. Finish.—The finished material shall be free from injurious defects and shall have a workmanlike finish.

140. Marking.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice bars and other small sections shall, when loaded for shipment, be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

141. Inspection.—The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

142. Rejection.—(*a*) Unless otherwise specified, any rejection based on tests made in accordance with § 132 shall be reported within five working days from the receipt of samples.

(*b*) Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

Rehearing.—Samples tested in accordance with § 132, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

SPECIAL METALS.

143. Cast-Iron.—Except where chilled iron is specified, castings shall be made of tough gray iron, with sulphur not over 0.10 per cent. They shall be true to pattern, out of wind and free from flaws and excessive shrinkage. If tests are demanded they shall be made on the "Arbitration Bar" of the American Society for Testing Materials, which is a round bar, 1¼ in. in diameter and 15 in. long. The transverse test shall be on a supported length of 12 in. with load at middle. The minimum breaking load so applied shall be 2,900 lb., with a deflection of at least $\frac{1}{10}$ in. before rupture.

144. Wrought-Iron Bars.—Wrought-iron shall be double-rolled, tough, fibrous and uniform in character. It shall be thoroughly welded in rolling and be free from surface defects. When tested in specimens of the form of Fig. 1, or in full-sized pieces of the same length, it shall show an ultimate strength of at least 50,000 lb. per sq. in., an elongation of at least 18 per cent in 8 in., with fracture wholly fibrous. Specimens shall bend cold, with the fiber through 135°, without sign of fracture, around a pin the diameter of which is not over twice the thickness of the piece tested. When nicked and bent the fracture shall show at least 90 per cent fibrous.

TIMBER.

145. Timber.—The timber shall be strictly first-class spruce, white pine, Douglas fir, Southern yellow pine, white oak, or other approved timber. Timber piles shall preferably be white, post or burr oak, Douglas fir, longleaf pine, tamarack, white or red cedar, chestnut, redwood or cypress.

146. General Requirements.—All timber shall be cut from sound live trees, and shall be sawed to standard size. It must be close grained and solid, free from defects such as injurious ring shakes and cross grain, unsound or loose knots, knots in groups, large pitch pockets, decay or other defects that will impair its strength or fitness for the purpose intended.

147. Size of Sawed Timber.—All timber shall be sawed true and out of wind and shall, when dry, not measure scant in thickness more than the following:

Flooring and boards up to 1½ in. thick, may be scant $\frac{1}{16}$ in.

Planks and timbers, rough size, from 1¾ to 5¾ in. thick, may be scant ⅛ in.

Dimension timber, rough size, 6 in. thick and up, may be scant ¼ in. For example, a 12 in. × 12 in. timber may be 11¾ in. × 11¾ in.

148. Size of Dressed Timber.—When dressed timber more than 1½ in. in thickness is required, a reduction of ⅛ in. in thickness for each surface planed will be permitted in addition to the allowance in rough timber in § 147. For example a 12 in. × 12 in. timber S.4S. may be 11½ in. × 11½ in.

149. Dimension Timber.—Dimension timber when used for beams, stringers, caps, posts and sills shall show not less than 75 per cent heart on each of four faces, measured across the sides anywhere in the length of the piece. There shall be no loose knots, or knots greater than

2 in. in diameter, or one-quarter (¼) the width of the face of the stick in which they occur. Knots shall not be located in groups and no knot shall be nearer the edge of the stick than one-quarter (¼) the width of the face. When used for other purposes dimension timber shall be square edged with exception of 1-in. wane on one edge or ½-in. wane on two edges, and ring shakes shall not extend over one-eighth (⅛) the length of the piece.

150. **Flooring.**—Flooring shall preferably be yellow pine, maple or beech, as specified, and shall be furnished usually in lengths of 12 to 16 ft. and not over 4 in. face. The thickness of the flooring shall be the thickness of the finished material. Flooring shall be edge grained, kiln dried, matched, tongued and grooved, planed on the upper side, well manufactured so as to be free from planer's marks, splinters, etc. It shall show one face all heart and shall be free from knots, shakes, sap and pitch pockets.

151. **Sub-Floor Plank.**—Floor plank shall be square edged, shall show one face all heart and the other face and two edges shall show not less than seventy-five (75) per cent heart, measured across the face or sides measured anywhere in the length of the piece; and shall be free from loose knots, or sound knots more than 1½ in. in diameter.

152. **Piles.**—Piles shall be cut from sound, live trees, shall be straight, close grained and solid, free from defects such as injurious ring shakes, large and unsound or loose knots, decay or other defects that will materially impair the strength or durability. The diameter of round piles near the butt shall not be less than 12 in. nor more than 18 in., and at the tip of piles under 30 ft. not less than 8 in., nor less than 6 in. for piles more than 30 ft. long. Piles must be cut above the ground swell and must taper evenly from butt to tip. Short bends will not be allowed. A line drawn from the butt to the tip shall lie entirely within the body of the pile. All piles shall be cut square at their ends and shall be stripped of their bark.

PART VII. WORKMANSHIP.

153. **General.**—All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works.

154. **Straightening Material.**—Material shall be thoroughly straightened in the shop, by methods that will not injure it, before being laid off or worked in any way.

155. **Finish.**—Shearing shall be neatly and accurately done and all portions of the work exposed to view neatly finished.

156. **Rivets.**—The size of rivets, called for on the plans, shall be understood to mean the actual size of the cold rivet before heating.

157. **Rivet Holes.**—When general reaming is not required, the diameter of the punch for material not over ¾ in. thick shall be not more than $\frac{1}{16}$ in., nor that of the die more than ⅛ in. larger than the diameter of the rivet. The diameter of the die shall not exceed that of the punch by more than ¼ the thickness of the metal punched.

158. **Planing and Reaming.**—In medium steel over ¾ of an in. thick, all sheared edges shall be planed and all holes shall be drilled or reamed to a diameter of ⅛ of an in. larger than the punched holes, so as to remove all the sheared surface of the metal. Steel which does not satisfy the drifting test must have holes drilled.

159. **Punching.**—Punching shall be accurately done. Slight inaccuracy in the matching of holes may be corrected with reamers. Drifting to enlarge unfair holes will not be allowed. Poor matching of holes will be cause for rejection by the inspector.

160. **Assembling.**—Riveted members shall have all parts well pinned up and firmly drawn together with bolts before riveting is commenced. Contact surfaces to be painted (see § 191).

161. **Lacing Bars.**—Lacing bars shall have neatly rounded ends, unless otherwise called for.

162. **Web Stiffeners.**—Stiffeners shall fit neatly between flanges of girders. Where tight fits are called for the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

163. **Splice Plates and Fillers.**—Web splice plates and fillers under stiffeners shall be cut to fit within ⅛ in. of flange angles.

164. **Web Plates.**—Web plates of girders, which have no cover plates, shall be flush with the backs of angles or be not more than ¼ in. scant, unless otherwise called for. When web plates are spliced, not more than ¼ in. clearance between ends of plates will be allowed.

165. **Connection Angles.**—Connection angles for girders shall be flush with each other and correct as to position and length of girder. In case milling is required after riveting, the removal of more than $\frac{1}{16}$ in. from their thickness will be cause for rejection.

166. **Riveting.**—Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

167. Rivets shall look neat and finished, with heads of approved shape, full and of equal size. They shall be central on shank and grip the assembled pieces firmly. Recupping and calking will not be allowed. Loose, burned or otherwise defective rivets shall be cut out and replaced. In cutting out rivets great care shall be taken not to injure the adjacent metal. If necessary they shall be drilled out.

168. **Turned Bolts.**—Wherever bolts are used in place of rivets which transmit shear, the holes shall be reamed parallel and the bolts turned to a driving fit. A washer not less than $\frac{1}{4}$ in. thick shall be used under nut.

169. **Members to be Straight.**—The several pieces forming one built member shall be straight and fit closely together, and finished members shall be free from twists, bends or open joints.

170. **Finish of Joints.**—Abutting joints shall be cut or dressed true and straight and fitted close together, especially where open to view. In compression joints depending on contact bearing the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

171. **Field Connections.**—All holes for field rivets in splices in tension members carrying moving loads shall be accurately drilled to an iron templet or reamed while the connecting parts are temporarily put together.

172. **Eye-Bars.**—Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body with a silky fracture, when tested to rupture. The thickness of head and neck shall not vary more than $\frac{1}{16}$ in. from the thickness of the bar.

173. **Boring Eye-Bars.**—Before boring, each eye-bar shall be properly annealed and carefully straightened. Pin holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that, when placed together, pins $\frac{1}{32}$ in. smaller in diameter than the pin holes can be passed through the holes at both ends of the bars at the same time.

174. **Pin Holes.**—Pin holes shall be bored true to gage, smooth and straight; at right angles to the axis of the member and parallel to each other, unless otherwise called for. Wherever possible, the boring shall be done after the member is riveted up.

175. The distance center to center of pin holes shall be correct within $\frac{1}{32}$ in., and the diameter of the hole not more than 1/50 in. larger than that of the pin, for pins up to 5 in. diameter, and $\frac{1}{32}$ in. for larger pins.

176. **Pins and Rollers.**—Pins and rollers shall be accurately turned to gage and shall be straight and smooth and entirely free from flaws.

177. **Pilot Nuts and Field Rivets.**—At least one pilot and one driving nut shall be furnished for each size of pin for each structure; and field rivets 15 per cent plus 10 rivets in excess of the number of each size actually required.

178. **Screw Threads.**—Screw threads shall make tight fits in the nuts and shall be U. S. standard, except above the diameter of 1$\frac{3}{8}$ in., when they shall be made with six threads per in.

179. **Annealing.**—Steel, except in minor details, which has been partially heated shall be properly annealed.

180. **Steel Castings.**—All steel castings shall be annealed.

181. **Welds.**—Welds in steel will not be allowed.

182. **Bed Plates.**—Expansion bed plates shall be planed true and smooth. Cast wall plates shall be planed top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

183. **Shipping Details.**—Pins, nuts, bolts, rivets, and other small details shall be boxed or crated.

184. **Weight.**—The weight of every piece and box shall be marked on it in plain figures.

185. **Weight Paid For.**—The payment for pound price contracts shall be based on scale weights of the metal in the fabricated structure, including field rivets 15 per cent plus 10 rivets in excess of the number nominally required. The weight of the shop coat of paint, field paint, cement, fitting up bolts, pilot nuts, driving caps, boxes and barrels used for packing, and material used in supporting members on cars shall be excluded. If the scale weight is more than 2$\frac{1}{2}$ per cent under the computed weight it may be cause for rejection. The greatest allowable variation of the total scale weight of any structure from the weights computed from the approved shop drawings shall be 2 per cent. Any weight in excess of 2 per cent above the computed weight

shall not be paid for. The weights of rolled shapes and plates shall be computed on the basis of their normal weights and dimensions, as shown on the approved drawings, deducting for all copes, cuts and open holes. With plates the percentage of overrun given in these specifications shall be added. The weight of heads of shop driven rivets shall be included in the computed weight. The weights of castings shall be computed from the dimensions shown on the approved drawings, with an addition of 10 per cent for fillets and overrun.

Additional Specifications When General Reaming and Planing are Required.

186. **Planing Edges.**—Sheared edges and ends shall be planed off at least $\frac{1}{4}$ in.

187. **Reaming.**—Punched holes shall be made with a punch $\frac{3}{16}$ in. smaller in diameter than the nominal size of the rivets and shall be reamed to a finished diameter of not more than $\frac{1}{16}$ in. larger than the rivet.

188. **Reaming after Assembling.**—Wherever practicable, reaming shall be done after the pieces forming one built member have been assembled and firmly bolted together. If necessary to take the pieces apart for shipping and handling, the respective pieces reamed together shall be so marked that they may be reassembled in the same position in the final setting up. No interchange of reamed parts will be allowed.

189. **Removing Burrs.**—The burrs on all reamed holes shall be removed by a tool countersinking about $\frac{1}{16}$ in.

Painting in Shop.

190. **Painting.**—All steel work before leaving the shop shall be thoroughly cleaned from all loose scale and rust, and be given one good coating of pure boiled linseed oil or paint as specified, well worked into all joints and open spaces.

191. In riveted work, the surfaces coming in contact shall each be painted (with paint) before being riveted together.

192. Pieces and parts which are not accessible for painting after erection shall have two coats of paint.

193. The paint shall be a good quality of red lead or graphite paint, ground with pure linseed oil, or such paint as may be specified in the contract.

194. Machine finished surfaces shall be coated with white lead and tallow before shipment or before being put out into the open air.

Inspection and Testing at Mill and the Shops.

195. The manufacturer shall furnish all facilities for inspecting and testing weight and the quality of workmanship at the mill or shop where material is fabricated. He shall furnish a suitable testing machine for testing full-sized members if required.

196. **Mill Orders.**—The engineer shall be furnished with complete copies of mill orders, and no materials shall be ordered nor any work done before he has been notified as to where the orders have been placed so that he may arrange for the inspection.

197. **Shop Plans.**—The engineer shall be furnished with approved complete shop plans, and must be notified well in advance of the start of the work in the shop in order that he may have an inspector on hand to inspect the material and workmanship.

198. **Shipping Invoices.**—Complete copies of shipping invoices shall be furnished the engineer with each shipment.

199. The engineer's inspector shall have full access, at all times, to all parts of the mill or shop where material under his inspection is being fabricated.

200. The inspector shall stamp each piece accepted with a private mark. Any piece not so marked may be rejected at any time, and at any stage of the work. If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the engineer.

201. **Full Size Tests.**—Full size tests of any finished member shall be tested at the manufacturer's expense, and shall be paid for by the purchaser at the contract price less the scrap value, if the tests are satisfactory. If the tests are not satisfactory the material will not be paid for and the members represented by the tested member may be rejected.

ERECTION.

202. **Tools.**—The contractor shall furnish at his own expense all necessary tools, staging and material of every description required for the erection of the work, and shall remove the same when the work is completed.

All field connections in the trusses and framework shall be riveted. Connections of purlins and girts may be bolted.

The contractor shall put in place all stone bolts and anchors for attaching the steel work to the masonry. He shall drill all the necessary holes in the masonry, and set all bolts with neat Portland cement.

203. Field rivets shall preferably be driven by pneumatic riveters of approved make. A pneumatic bucker shall be used with a pneumatic riveter. Splices and field connections shall have 50 per cent of the holes filled with bolts and drift pins (of which one-fifth shall be drift pins) before riveting. Rivets in splices of compression chords shall not be driven until the abutting surfaces have been brought into contact throughout, and submitted to full dead load stress. Field riveting shall be done to the satisfaction of the engineer.

204. The erection will also include all necessary hauling from the railroad station, the unloading of the materials and their proper care until the erection is completed.

205. Whenever new structures are to replace existing ones, the latter are to be carefully taken down and removed by the contractor to some place where the material can be hauled away.

206. The contractor shall so conduct his work as not to interfere with traffic, interfere with the work of other contractors, or close any thoroughfare.

207. The contractor shall assume all risks of accidents and damages to persons and properties prior to the acceptance of the work.

208. The contractor must remove all falsework, piling and other obstructions or unsightly material produced by his operations.

209. The contractor shall comply with all ordinances or regulations appertaining to the work.

210. The erection shall be carried forward with diligence and shall be completed promptly.

PAINTING AFTER ERECTION.

211. **Painting.**—After the building is erected the metal work shall be thoroughly cleaned of mud, grease or other material, then thoroughly and evenly painted with two coats of paint of the kind specified by the engineer, mixed with linseed oil. All recesses which may retain water, or through which water can enter, must be filled with thick paint or some waterproof cement before final painting. The different coats of paint must be of distinctly different shades or colors, and one coat must be allowed to dry thoroughly before the second coat is applied. All painting shall be done with round brushes of the best quality obtainable on the market. The paint shall be delivered on the work in the manufacturer's original packages and be subject to inspection. If tests made by the inspector shows that the paint is adulterated, the paint will be rejected and the contractor shall pay the cost of the analyses, and shall scrape off and thoroughly clean and repaint all material that has been painted with the condemned paint. The paint shall not be thinned with anything whatsoever; in cold weather the paint may be thinned by heating under the direction of the inspector. No turpentine nor benzine shall be allowed on the work, except by the permission of the inspector, and in such quantity as he shall allow. The inspector shall be notified when any painting is to be done by the contractor, and no painting shall be done until the inspector has approved the surface to which the paint is to be applied. Paint shall not be applied out of doors in freezing, rainy, or misty weather, and all surfaces to which paint is to be applied shall be dry, clean and warm. **In cool weather the paint may be thinned by heating, and this may be required by the inspector.**

REFERENCES.—For data on windows and glazing; paints and painting; foundations, and additional data and examples of roof trusses and steel mill buildings, see the author's "The Design of Steel Mill Buildings." This book also contains a full treatment of algebraic and graphic statics; and the calculation of stresses in simple framed structures, in the transverse bent, the two-hinged arch, and other statically undeterminate structures; also contains 40 problems in algebraic and graphic statics illustrating the methods of calculating the stresses in roof trusses and other framed structures.

CHAPTER II.

STEEL OFFICE BUILDINGS.

Skeleton Construction.—Skeleton construction is a building where all external and internal loads and stresses are transferred from the top of the building to the foundations by a skeleton or framework of steel or reinforced concrete. In steel skeleton construction the framework consists of columns, floorbeams, girders, trusses, and diagonal and transverse bracing. The steel trusses have riveted connections and all connections in the steel framework should be riveted.

Fire Resisting Construction.—To protect the structural steel from fire the framework is covered with materials that are slow heat conducting or "fireproof material." The steel framework may be fireproofed with reinforced concrete, brick, tiles of burnt clay, or terra cotta. The windows on exposed sides and elevator enclosures are glazed with wire glass set in metal frames or are protected with fire shutters. Doors and other exposed openings are protected with fire doors or shutters. The interior finish, doors, etc. should be of metal and every precaution should be taken to prevent the spread of fire. Reinforced concrete fireproofing is usually made of the following thickness: For columns, trusses, girders or other very important members at least 2 inches of concrete outside of the metal reinforcement; for ordinary beams or long span floor slabs or arches, 1½ inches of concrete outside of the reinforcement, and for short span floor arches and slabs, partitions and walls at least 1 inch outside the metal reinforcement. Fireproofing of brick, tile or terra cotta is usually made with a thickness of not less than 4 inches for columns and the main framework. Metal flanges should be protected with not less than 2 inches of fireproofing at any point.

TABLE I.

WEIGHTS OF BUILDING MATERIALS, ETC.
POUNDS PER CUBIC FOOT.

Material.	Weight.	Material.	Weight.
Brick, pressed and paving	150	Hemlock	25
" common building	120	White pine	25
" soft building	100	Douglas fir	30
Granite	170	Yellow pine	40
Marble	170	White oak	50
Limestone	160	Mortar	100
Sandstone	150	Stone concrete	150
Cinders	40	Cinder "	110
Slag	160–180	Common brick work	100–120
Granulated furnace slag	53	Rubble masonry, sandstone	130–140
Gravel	120	" " limestone	140
Slate	175	" " granite	150
Sand, clay and earth (dry)	100	Ashlar " sandstone	140–150
" " " " (moist)	120	" " limestone	150
Coal ashes	45	" " granite	165
Paving asphaltum	100	Cast iron	450
Plaster of Paris	140	Wrought iron	480
Glass	160	Steel	490
Water	62½	Lead	711
Snow, freshly fallen	5	Copper, rolled	490
" packed	12	Brass	523
" wet	50	Plaster, ceiling 10 to 15 lb. per sq. ft.	
Spruce	25		

For details and data on fireproofing and fireproofing materials, see Freitag's "Fire Prevention and Fire Protection," and Kidder's "Architects and Builders Pocket Book."

LOADS.—The loads coming on office buildings may be grouped under the following headings: (1) dead loads; (2) live loads; (3) wind loads; (4) snow loads; (5) miscellaneous loads.

Dead Load.—The "dead load" includes the weight of the structure, and other permanent fixtures and machines. A formula for the weight of roof trusses is given in Chapter I. The weights of materials are given in Table I. The actual weights of all dead loads should be calculated. The minimum weight of a fireproof floor should be taken at not less than 75 lb. per sq. ft. of floor surface. In office buildings a minimum of 10 lb. per sq. ft. should be added for movable partitions.

WEIGHT OF STEEL IN TALL BUILDINGS.—The weight of the steel framework for tall steel buildings varies with the height, the column spacing, the floor loads and other conditions. The weights of steel per cubic foot for several tall steel buildings are given in Table II. In calculating the weight per cubic foot only the part of the building above the curb was considered.

TABLE II.

WEIGHT OF STEEL IN TALL BUILDINGS, POUNDS PER CUBIC FOOT.

Building.	Plan Sq. Ft.	Height. Stories.	Height. Ft.	Weight of Steel, Lb. per Cu. Ft.	Reference.
Park Row Building, New York..	15,000	26	307	3.6	Eng. News, Oct. 8, 1896
Hotel Astor (addition), New York....................	21,306	9	...	2.6	Eng. Record, Oct. 14, 1911
Banker's Trust Building, New York....................	9,018	39	543	3.1	Eng. Record, Feb. 11, 1911
Underwood Building, New York.	3,952	18	220	2.6	Eng. Record, April 1, 1911
Hotel Rector, New York.......	13,231	13	...	2.3	Eng. Record, May 27, 1911
Woolworth Building, New York.	31,000	55	775	3.0	Eng. Record, May 27, 1911
Municipal Building, New York..	42,686	..	580	3.6	Eng. News, July 27, 1911
Poole Bros. Printing, Chicago...	5,000	7	...	2.1	Eng. News, July 25, 1912
Merchants & Mfgs. Exchange, New York................	55,000	12	...	2.8	Eng. Record, May 11, 1912
Hotel McAlpin, New York.....	39,500	25	309	2.0	Eng. Record, Mar. 30, 1912
Curtis Building, Philadelphia...	94,000	10	176	3.0	Eng. Record, July 9, 1910
Office Building, Denver........	7,500	12	145	2.8	Designed by the author

Live Loads.—The live loads on floors are commonly given in pounds per square foot. The minimum live loads in pounds per square foot as required by the buildings laws of several cities are given in Table III.

Mr. C. C. Schneider, M. Am. Soc. C. E., in his "General Specifications for Structural Work of Buildings" gives the following requirements for live loads on floors.

"Table IV gives the 'live' load on floors, to be assumed for different classes of buildings. These loads consist of: (*a*) A uniform load per square foot of floor area; (*b*) A concentrated load which shall be applied to any point of the floor; (*c*) A uniform load per linear foot for girders. The maximum result is to be used in calculations. The specified concentrated loads shall also apply to the floor construction between the beams for a length of 5 ft."

TABLE III.

Floors and Roofs.

Minimum Live Loads, Pounds per Square Foot.

By Building Laws of Various Cities.

Carnegie Steel Company.

Description of Building	New York, 1917	Chicago, 1919	Philadelphia, 1919	St. Louis, 1917	Boston, 1919	Cleveland, 1920	Baltimore, 1908	Pittsburgh, 1914	Cincinnati, 1917
Floors for Rooms									
Apartments and Dwellings.	40	40	70	50	50	70a	60	50	40
Asylums, Hospitals, etc. . .	100	50	70	50	50c			70	40
Detention Buildings, etc. .	100	50			50e	80			60
Factories:									
Light manufacture.	120d	100d	120d	100d	125d		125d	125d	100d
Heavier manufacture. . .			150d	150d	250d		175d		150d
Hotels, Lodging Houses . .	40	50	70	50	50c	70	60	70	40b
Office Buildings, etc.	60	50	100	60b	75b	70b	75b	70	50b
Public Buildings:									
Municipal Buildings. . . .	100				75c	100			100
Churches	100	100	120	75	100	80	75	125	100
Libraries, Museums. . . .	100				100	125		200	
Theaters.	100	100	120	100	100	80	75	125	100
Schools, Colleges, etc.	75	75		75	50	70	75	70	60
Stores, light goods	120	100	120	100	125	100b	125	125	100
" heavier goods.			150	150	250		175		150
Warehouses.			150	150	250		250	200	150
Floors for Assembly Halls, etc.									
Auditoriums, fixed seats . .	100	100	120	100	100	80	75	125	100
" movable seats	100	100	120	100	100	125	125	125	100
Armories, Dance Halls, etc.	100	100			100	150		150	150
Miscellaneous									
Garages, Stables.	120	100e		100	150e	150e	100		75
Corridors, Hallways.	100	100		100	75f	70g			80g
Stairways, Fire Escapes . .	100	100		100	75f	100h			80g
Sidewalks.	300				250	200	200		300
Roofs:									
Flat, slope up to 20° (1/8)	40	25	30i	30	40	35i	40	50k	25
Steep, slope over 20° (1/8)	30	25	30i		25j	30i	20	50k	25
Wind Pressure	30l	20	30m	30	10-20n	20o	30	25	20p

a Dwellings, Cleveland, 60.
b First floors: St. Louis, 100; Boston, 125; Cleveland, 125; Baltimore, 150; Cincinnati, 100.
c Public floors of Hospitals, Hotels, Public Buildings, etc.: Boston, 100.
d Floor loads do not include the weight or the impact load of machinery.
e Garages, private: Chicago, 40; Boston, 75; Garages; public, upper floors: Cleveland, 100; Stables: Cleveland, 80.
f Corridors, stairways, etc., for Assembly Halls, Armories, etc.: Boston, 100.
g Except in Dwellings where floor loads are less.
h Stairways, etc., for Apartment Houses, 80; Dwellings, 60.
i Loads per square foot of superficial roof area; other roof loads are for the projected area.
j Loads include Wind Pressure: 10 pounds up to 3/8 slope, 15 up to 1/4 slope, 20 over 1/4 slope.
k Dead and live load; snow load 25 pounds, reduced 1 pound each degree between 20° and 45°.
l For buildings over 150 feet high, or where height is over 4 times least horizontal dimension.
m Wind pressure for high buildings in built-up districts: 25 pounds at tenth story, 2 1/2 pounds less for each story below and 2 1/2 pounds more for each story above, up to 35 pounds.
n For buildings 40 feet high, 10 pounds; up to 80 feet, 15 pounds; over 80 feet, 20 pounds.
o Wind pressure on curtain walls, 30 pounds.
p For buildings over 100 feet high, or where height is over 3 times the average width of base.

TABLE IV.

TABLE OF LIVE LOADS, SCHNEIDER'S SPECIFICATIONS.

Classes of Buildings.	Live Loads in Pounds.		
	Distributed Load.	Concentrated Load.	Load per Linear Ft. of Girder.
Dwellings, hotels, apartment-houses, dormitories, hospitals	40	2 000	500
Office buildings, upper stories	50	5 000	1 000
Schoolrooms, theater galleries, churches	60	5 000	1 000
Ground floors of office buildings, corridors and stairs in public buildings	80	5 000	1 000
Assembly rooms, main floors of theaters, ballrooms, gymnasia, or any room likely to be used for drilling or dancing	Floor 100 Columns 50	5 000	1 000
Ordinary stores and light manufacturing, stables and carriage-houses	80	8 000	1 000
Sidewalks in front of buildings	300	10 000	1 000
Warehouses and factories	from 120 up	Special	Special
Charging floors for foundries	" 300 "	"	"
Power houses, for uncovered floors	" 200 "	The actual weights of engines, boilers, stacks, etc., shall be used, but in no case less than 200 lb. per sq. ft.	

"If heavy concentrations, like safes, armatures, or special machinery, are likely to occur on floors, provision should be made for them. For structures carrying traveling machinery, such as cranes, conveyors, etc., 25 per cent shall be added to the stresses resulting from such live load, to provide for the effects of impact and vibration."

Mr. Schneider's method for live loads is the most rational method yet proposed. In the design of floor slabs when using this method the author has used an equivalent distributed load equal to twice the distributed loads in Table IV, and has omitted the concentrated load and load per lineal foot of girders.

The floor loads on warehouses and the recommended floor loads per sq. ft. have been tabulated by the American Bridge Company in Table V.

Wind Loads.—The wind loads required by different cities are given in Table III.

Schneider's specifications for wind load are as follows:

"The wind pressure shall be assumed as acting in any direction horizontally: First.—At 20 lb. per sq. ft. on the sides and ends of buildings and on the actually exposed surface, or the vertical projection of roofs; Second.—At 30 lb. per sq. ft. on the total exposed surfaces of all parts composing the metal framework. The framework shall be considered an independent structure, without walls, partitions or floors."

Additional data on wind loads are given in Chapter I.

Snow Loads.—The snow loads on roofs are given in Fig. 1, Chapter I.

Schneider's specifications require "A snow load of 25 lb. per sq. ft. of horizontal projection of the roof for all slopes up to 20 degrees; this load to be decreased 1 lb. for every degree of increase of slope up to 45 degrees, above which no snow load is to be considered. The above snow loads are minimum values for localities, where snow is likely to occur. In severe climates these snow loads should be increased in accordance with the actual conditions existing in these localities."

TABLE V.

FLOOR LOADS.

CONTENTS OF STORAGE WAREHOUSES.

American Bridge Company.

Material.	Weights per Cubic Foot of Space, Pounds.	Height of Pile, Feet.	Weights per Square Foot of Floor, Pounds.	Recommended Live Loads, Pounds per Square Foot.
Groceries, Wines, Liquors, Etc.				
Beans, in bags	40	8	320	
Canned Goods, in cases	58	6	348	
Coffee, Roasted, in bags	33	8	264	
Coffee, Green, in bags	39	8	312	
Dates, in cases	55	6	330	
Figs, in cases	74	5	370	
Flour, in barrels	40	5	200	
Molasses, in barrels	48	5	240	
Rice, in bags	58	6	348	250 to 300
Sal Soda, in barrels	46	5	230	
Salt, in bags	70	5	350	
Soap Powder, in cases	38	8	304	
Starch, in barrels	25	6	150	
Sugar, in barrels	43	5	215	
Sugar, in cases	51	6	306	
Tea, in chests	25	8	200	
Wines and Liquors, in barrels	38	6	228	
Dry Goods, Cotton, Wool, Etc.				
Burlap, in bales	43	6	258	
Coir Yarn, in bales	33	8	264	
Cotton, in bales, compressed	18	8	144	
Cotton Bleached Goods, in cases	28	8	224	
Cotton Flannel, in cases	12	8	96	
Cotton Sheeting, in cases	23	8	184	
Cotton Yarn, in cases	25	8	200	
Excelsior, compressed	19	8	152	
Hemp, Italian, compressed	22	8	176	
Hemp, Manila, compressed	30	8	240	200 to 250
Jute, compressed	41	8	328	
Linen Damask, in cases	50	5	250	
Linen Goods, in cases	30	8	240	
Linen Towels, in cases	40	6	240	
Sisal, compressed	21	8	168	
Tow, compressed	29	8	232	
Wool, in bales, compressed	48	.	...	
Wool, in bales, not compressed	13	8	104	
Wool, Worsteds, in cases	27	8	216	

Material.	Weights per Cubic Foot of Space, Pounds.	Height of Pile, Feet.	Weights per Square Foot of Floor, Pounds.	Recommended Live Loads, Pounds per Square Foot.
Building Materials				
Cement, Natural	59	6	354	
Cement, Portland	73	6	438	300 to 400
Lime and Plaster	53	5	265	
Hardware, Etc.				
Door Checks	45	.	...	
Hinges	64	.	...	
Locks, in cases, packed	31	.	...	
Sash Fasteners	48	.	...	
Screws	101	.	...	
Sheet Tin, in boxes	278	2	556	300 to 400
Wire Cables, on reels	...	.	425	
Wire, Insulated Copper, in coils	63	5	315	
Wire, Galvanized Iron, in coils	74	4½	333	
Wire, Magnet, on spools	75	6	450	
Drugs, Paints, Oil, Etc.				
Alum, Pearl, in barrels	33	6	198	
Bleaching Powder, in hogsheads	31	3⅓	102	
Blue Vitriol, in barrels	45	5	226	
Glycerine, in cases	52	6	312	
Linseed Oil, in barrels	36	6	216	
Linseed Oil, in iron drums	45	4	180	
Logwood Extract, in boxes	70	5	350	
Rosin, in barrels	48	6	288	200 to 300
Shellac, Gum	38	6	228	
Soda Ash, in hogsheads	62	2¾	167	
Soda, Caustic, in iron drums	88	3⅓	294	
Soda, Silicate, in barrels	53	6	318	
Sulphuric Acid	60	1⅔	100	
White Lead Paste, in cans	174	3½	610	
White Lead, dry	86	4¾	408	
Red Lead and Litharge, dry	132	3¾	495	
Miscellaneous				
Glass and Chinaware, in crates	40	8	320	
Hides and Leather, in bales	20	8	160	
Hides, Buffalo, in bundles	37	8	296	
Paper, Newspaper, and Strawboards	35	6	210	300
Paper, Writing and Calendared	60	6	360	
Rope, in coils	32	6	192	

Minimum Roof Loads.—Schneider's specifications contain the following:

"In climates corresponding to that of New York, ordinary roofs, up to 80 ft. span, shall be proportioned to carry the minimum loads in Table VI, per square foot of exposed surface, applied vertically, to provide for dead, wind and snow loads combined:

TABLE VI.

MINIMUM LOADS ON ROOFS.

Gravel or Composition Roofing	On boards, flat slope, 1 to 6, or less	50 lb.
	On boards, steep slope, more than 1 to 6	45 "
	On 3-in. flat tile or cinder concrete	60 "
Corrugated sheeting, on boards or purlins		40 "
Slate	On boards or purlins	50 "
	On 3-in. flat tile or cinder concrete	65 "
Tile, on steel purlins		55 "
Glass		45 "

"For roofs in climates where no snow is likely to occur, reduce the foregoing loads by 10 lb. per sq. ft., but no roof or any part thereof shall be designed for less than 40 lb. per sq. ft."

LIVE LOADS ON COLUMNS.—Schneider's specifications require that:

"For columns, the specified uniform live loads per square foot, Table IV, shall be used, with a minimum of 20,000 lb. per column.

"For columns carrying more than five floors, these live loads may be reduced as follows:

"For columns supporting the roof and top floor, no reduction;

"For columns supporting each succeeding floor, a reduction of 5 per cent of the total live load may be made until 50 per cent is reached, which reduced load shall be used for the columns supporting all remaining floors."

The Chicago Building Ordinance (1911) requires that live loads on walls, columns and piers be taken as follows:

"(a) The full live load (see Table III) on roofs of all buildings shall be taken on walls, piers, and columns.

"(b) The walls, piers and columns of all buildings shall be designed to carry the full dead loads and not less than the proportion of the live load given in Table VII.

TABLE VII.

PERCENTAGE OF LIVE LOAD FOR COLUMNS.

Chicago Building Ordinance (1911).

Floor	17	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1
17	85 per cent																
16	80	85															
15	75	80	85														
14	70	75	80	85													
13	65	70	75	80	85												
12	60	65	70	75	80	85											
11	55	60	65	70	75	80	85										
10	50	55	60	65	70	75	80	85									
9	50	50	55	60	65	70	75	80	85								
8	50	50	50	55	60	65	70	75	80	85							
7	50	50	50	50	55	60	65	70	75	80	85						
6	50	50	50	50	50	55	60	65	70	75	80	85					
5	50	50	50	50	50	50	55	60	65	70	75	80	85				
4	50	50	50	50	50	50	50	55	60	65	70	75	80	85			
3	50	50	50	50	50	50	50	50	55	60	65	70	75	80	85		
2	50	50	50	50	50	50	50	50	50	55	60	65	70	75	80	85	
1	50	50	50	50	50	50	50	50	50	50	55	60	65	70	75	80	85

"(c) The proportion of the live load on walls, piers, and columns on buildings more than seventeen stories in height shall be taken in same ratio as the above table.

"(d) The entire dead load and the percentage of live load on basement columns, piers and walls shall be taken in determining the stress in foundations."

LOADS ON FOUNDATIONS.—Schneider's specifications require that:

"The live loads on columns shall be assumed to be the same as for the footings of columns. The areas of the bases of the columns shall be proportioned for the dead load only. That foundation which receives the largest ratio of live to dead load shall be selected and proportioned for the combined dead and live loads. The dead load on this foundation shall be divided by the area thus found and this reduced pressure per square foot shall be the permissible working pressure to be used for the dead load for all foundations."

PRESSURE ON FOUNDATIONS.—The following allowable pressures may be used in the absence of definite data. No important structure should be built without the making of careful tests of the bearing power of the soil upon which it is to rest.

The loads on foundations should not exceed the following in tons per square foot:

Ordinary clay and dry sand mixed with clay	2
Dry sand and dry clay	3
Hard clay and firm, coarse sand	4
Firm, coarse sand and gravel	5
Shale rock	8
Hard rock	20

For all soils inferior to the above, such as loam, etc., never more than one ton per square foot.

The Chicago Building Ordinance (1911) requires that:

"(a) If the soil is a layer of pure clay at least fifteen feet thick, without admixture of any foreign substance other than gravel it shall not be loaded to exceed 3,500 lb. per sq. ft. If the soil is a layer of pure clay at least fifteen feet thick and is dry and thoroughly compressed, it may be loaded not to exceed 4,500 lb. per sq. ft.

"(b) If the soil is a layer of firm sand fifteen feet or more in thickness, and without admixture of clay, loam or other foreign substance, it shall not be loaded to exceed 5,000 lb. per sq. ft.

"(c) If the soil is a mixture of clay and sand, it shall not be loaded to exceed 3,000 lb. per sq. ft.

"Foundations shall in all cases extend at least four feet below the surface of the ground upon which they are built, unless footings rest on bed rock."

PRESSURE ON MASONRY.—The allowable stresses in masonry and pressures of beams, girders, column bases, etc. on masonry as given in Table VIII represent good practice.

TABLE VIII.

ALLOWABLE STRESSES IN MASONRY AND PRESSURES OF BEARING PLATES.

Kind of Masonry.	Safe Stresses in Masonry, Lb. per Sq. In.	Safe Pressures of Walls, Plates and Columns on Masonry, Lb. per Sq. In.
Common Brick, Portland Cement Mortar	170	250
Hard burned brick, Portland Cement Mortar	210	300
Rubble Masonry, Portland Cement Mortar	170	250
First Class Masonry, Sandstone	280	350
First Class Masonry, Crystallized Sandstone	400	600
First Class Masonry, Limestone	300	500
First Class Masonry, Granite	400	600
Portland Cement Concrete, 1-2-4	400	600
Portland Cement Concrete, 1-3-5	300	400

BEARING POWER OF PILES.—The maximum load carried by a pile should not exceed 40,000 lb. Piles should be driven not less than 10 ft. in hard material, nor less than 20 ft. in soft material if the pile is to be loaded to full bearing. The safe load should not exceed that given by the Engineering News formula (1), Chapter XIV.

THICKNESS OF WALLS.—The minimum thickness of curtain walls in steel skeleton buildings should be 12 in. for brick or concrete and 8 in. for reinforced concrete.

Schneider's specifications give the following empirical rule for calculating the thickness of walls in buildings several stories in height.

"The minimum thickness of walls will be given by the formula

$$t = L/4 + (H_1 + H_2 + \cdots + H_n)/6$$

where t = minimum thickness of wall in inches, L = unsupported length in feet, which shall be assumed as not less than 24 ft.; and H_1, H_2, H_3, etc. the heights of stories in feet beginning at the top. Cellar walls are to be 4 in. thicker than the first story walls."

The Chicago Building Ordinance (1911) contains the following:

"(a) Brick, stone, and solid concrete walls, except as otherwise provided, shall be of the thickness in inches indicated in the following table:"

THICKNESS OF WALLS.
Chicago Building Ordinance (1911).

	Basement.	Stories.											
		1	2	3	4	5	6	7	8	9	10	11	12
One-story	12	12											
Two-story	16	12	12										
Three-story	16	16	12	12									
Four-story	20	20	16	16	12								
Five-story	24	20	20	16	16	16							
Six-story	24	20	20	20	16	16	16						
Seven-story	24	20	20	20	20	16	16	16					
Eight-story	24	24	24	20	20	20	16	16	16				
Nine-story	28	24	24	24	20	20	20	16	16	16			
Ten-story	28	28	28	24	24	24	20	20	20	16	16		
Eleven-story	28	28	28	24	24	24	20	20	20	16	16	16	
Twelve-story	32	28	28	28	24	24	24	20	20	20	16	16	16

WATERPROOFING.—For methods of waterproofing walls, floors, etc., see methods of waterproofing bridge floors in Chapter IV.

CALCULATION OF WIND LOAD STRESSES.—(1) The wind load on the sides of the steel frame in a building in which the wind bracing is all in the outside walls of the building will be carried to the ends of the building by means of bracing in the plane of each floor or by the floor slabs where the floors are made of reinforced concrete, and the loads will then be transferred to the foundations by means of bracing in the planes of the ends of the building. In calculating the stresses in the bracing in the end panels it is usual to assume that the wind load carried by each braced bent, consisting of two columns, together with the floor girders and wind bracing, is equal to the total wind load divided by the number of braced panels in the plane. This was the method used in calculating the stresses in the Singer Tower, New York. (2) As usually constructed the interior columns have brackets and only part of the wind load will be transferred to the ends or sides of the building, the remainder of the wind load will be transferred to the foundations by portal action and flexure in the columns and beams. It is not possible to determine the proportion of the wind load that will be taken by the main framework and by the ends of the building, as the stresses in the framework are statically indeterminate. During erection and before the floors have been put in place, or with types of floors which do not increase the rigidity of the building in horizontal planes, the wind loads will all be taken by the framework normal to the side of the building upon which the wind blows. This wind load is commonly taken as 30 lb. per sq. ft. of all framework exposed. When rigid floors have been put in place and the building is completed the wind load will be taken by the end transverse frames and the intermediate transverse frames, in proportion to the relative rigidity of the two frameworks. In a long narrow building with efficient wind bracing in the intermediate framework, practically all the wind load will be taken directly to the foundations by the transverse intermediate bents; while in the direction of the length of the building, practically all the wind load will be carried by the bracing in the sides of the building. For a building as long as wide with rigid floors and efficient transverse framework

and efficient wind bracing in the ends and sides of the building, it would appear reasonable to assume that in the completed building one-half the wind load will be taken by the intermediate transverse framework, and one-half will be transferred by means of the floors to the ends of the building and then transferred to the foundations by means of wind bracing in the ends of the building. The author's specifications permit reinforced concrete floors to be considered as assisting in transferring wind loads in finished buildings, but most specifications require that the steel framework be required to carry all the wind loads in the completed structure.

The transverse intermediate framework usually consists of columns and floor girders, in which the floor girders have brackets or knee braces at the ends to increase the rigidity of the framework. It will be seen that it is not only impossible to calculate the amount of wind load that is taken by each intermediate transverse framework, but that the intermediate transverse framework is itself statically indeterminate. In addition to being statically indeterminate it is not possible to determine the sizes of the columns and floor girders until after the wind stresses are determined. With a given framework in which the sizes of the members and the loads are given the stresses may be calculated by taking into account the deformations of the structure.

Exact Methods.—The exact stresses in a transverse bent of a steel office building may be calculated by taking into account the deformations of the members. All methods for the calculation of the exact stresses in a tall frame building are based on the "Theory of Work" but have different names due to the methods of applying the equations of equilibrium. The stresses are most easily calculated by the "Slope Deflection Method" described in the author's Steel Mill Buildings, Fourth Edition. For the calculation of the moments in a steel frame office building by the "Slope Deflection Method," see "Wind Stresses in Steel Frames of Office Buildings," by Wilson and Maney, Bulletin No. 80, Engineering Experiment Station of the University of Illinois. For the calculation of the moments in a steel frame office building by the "Work Method," see article by Mr. Albert Smith, Journal Western Society of Engineers, April, 1915.

Approximate Methods.—To calculate the true stresses in the steel frame of an office building not only requires a very large amount of labor but also requires that the column and girder sections be known. Three approximate method for calculating the wind stresses in steel frame office buildings are described by Mr. R. Fleming in Engineering News, March 13, 1913. The third method described by Mr. Fleming was given in the earlier editions of this book. While this method gives very satisfactory results, the method gives unequal bending moments in the two ends of girders, and is limited to a transverse framework with four bays.

The following method is essentially the second method proposed by Mr. Fleming and is the method given in the author's "The Design of Steel Mill Buildings," page 351. In this method the transverse framework is assumed as divided into as many multiple portals as there are bays, by passing a vertical plane through each interior column. The external horizontal wind loads are assumed as divided equally between the portals if the girder spans or bays are equal, or as proportional to the span lengths if the bays are unequal. For equal bays the shears on the inside columns are equal to double the shears on the outside columns. The points of contraflexure in the columns are assumed as midway between the centers of horizontal girders. The points of contraflexure in the horizontal girders come at the centers of the bays. The vertical wind loads will all be taken by the outside columns. The stresses in a steel frame are very easily calculated by this method, as will be shown in the following problem.

Problem.—In the two story double bay frame shown in the upper right hand corner of Fig. 1, assume that the frame is divided into two bents, and that each bent carries one-half of the total load as shown. In both bents the shear in the columns will be assumed as equal. There will be a point of contraflexure at the center of each girder. There will be a point of contraflexure in each column near the center of each story height (the point of contraflexure will be assumed as coming midway between the horizontal girders). Each bent is now statically determinate and the bending moments and the stresses may be calculated. The moments, shears and stresses in the middle columns are added algebraically. It will be noted (1) that the shear in the center column is twice the shear in the outside columns, and (2) that there will be no vertical stresses in the middle column.

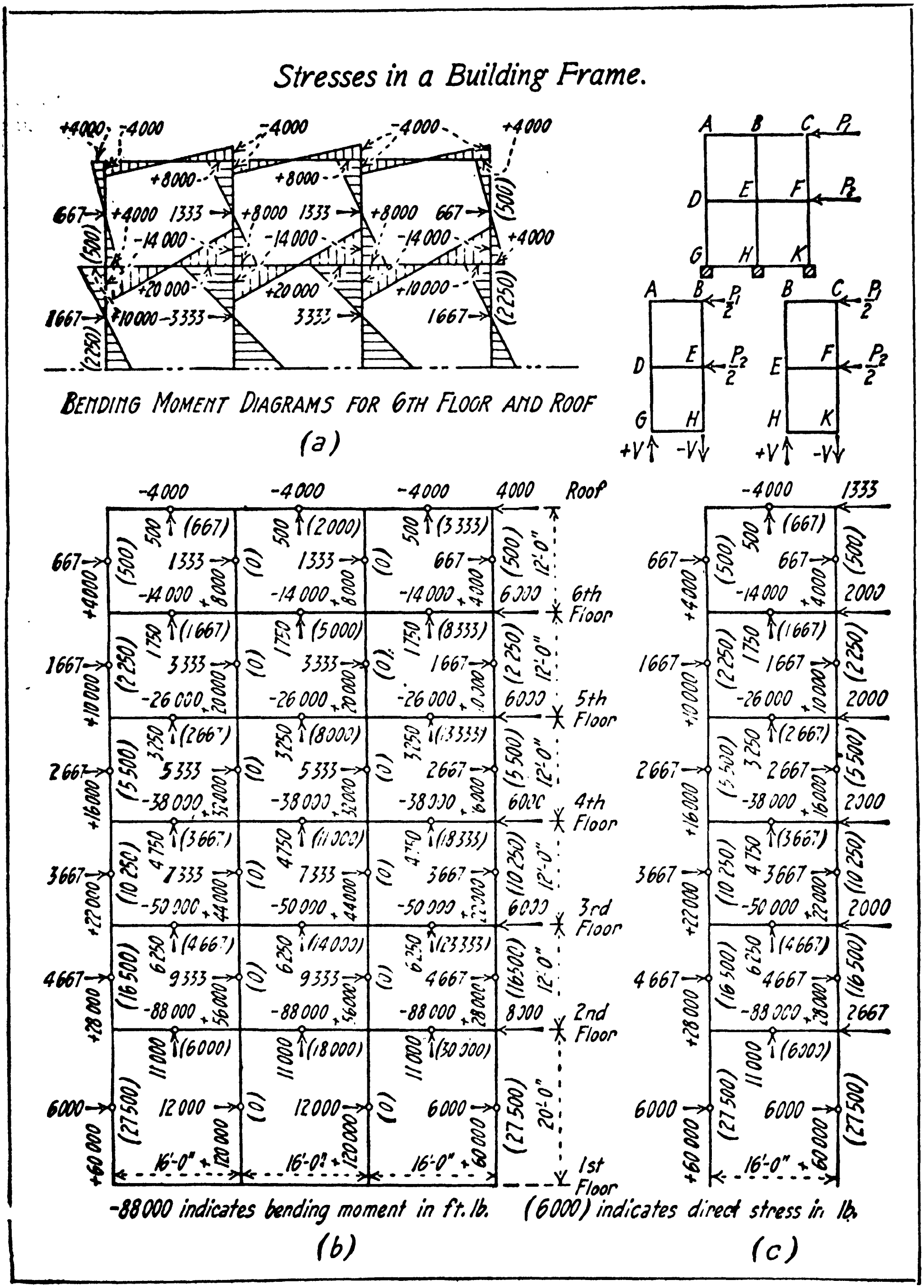


FIG. 1. WIND STRESSES IN A STEEL FRAME BUILDING.

The moments and stresses in the main steel frame were calculated by dividing the frame into three six-story bents, each of which carries one-third of the total wind load at each floor. The stresses are calculated in one bent as shown in (c), and the moments and stresses in the steel frame are shown in (b). Referring to (a) it will be seen that the shear in any interior column in any story will be twice the shear in each outside column. The vertical shear in the girders in all bents at one floor will be equal. The maximum moment in each column will be equal to the horizontal shear in the column multiplied by the distance from the point of contraflexure to the joint (6 ft.); while the maximum moment in each girder will be equal to the vertical shear in the girder multiplied by the distance from the point of contraflexure to the end of the girder (8 ft.). The sum of the bending moments about any joint will be equal to zero. If the bays are unequal the horizontal wind loads taken by the separate one-bay bents are to be taken directly proportional to the spans of the bays. This will make the vertical shears the same in all bays in any story.

Eccentric Riveted Connections.—For the calculation of the stresses in eccentric riveted connections, see Chapter XVII, and Table 118a, Part II.

ALLOWABLE STRESSES.—The general practice in designing tall steel frame buildings has been to use allowable stresses based on a tensile stress of 16,000 lb. per sq. in. Table IX gives the column formulas and allowable stresses to be used in designing columns and struts as specified by standard formulas and by building ordinances of several cities. While this table was printed in the first edition of this book it still represents conservative practice.

The allowable stresses given in the Standard Specifications for Steel Buildings, printed in the latter part of this chapter, adopted June 1, 1923, by the American Institute of Steel Construction, are based on an allowable tensile stress of 18,000 lb. per sq. in. These specifications assume that all loads shall be provided for, that proper provision shall be made for impact, that high grade structural steel shall be used, and that workmanship in shop and field shall be first class. The column formula is of the Rankine type and is based on a unit stress of 18,000 lb. per sq. in. with a maximum stress of 15,000 lb. per sq. in. The column formula is given in § 5 of the specifications and in Fig. 28.

Between the limits of l/r of 60 and 120 this column formula may be replaced by the formula

$$P = 20{,}000 - 83\, l/r$$

with a maximum stress of 15,000 lb. per sq. in.

The allowable stresses in the unsupported flanges of beams and girders are given in § 5 and in Fig. 29.

The shear in webs and design of stiffness are given in § 7 and in Fig. 30.

At the present time, January, 1924, several cities have adopted the allowable stresses given in the Standard Specifications for Steel Buildings of the American Institute of Steel Construction and nine or ten other cities permit its use. This specification represents the most recent practice and gives promise of being generally adopted.

Comparison of Compression Formulas.—The standard formula for the design of compression members adopted by the Am. Ry. Eng. Assoc., is used by the author in his "Specifications for Steel Frame Buildings" in Chapter I, and by the building ordinance of Chicago. The A. R. E. A. formula is

$$P = 16{,}000 - 70l/r \qquad (1)$$

where P = unit stress in lb. per sq. in.; l = length and r = least radius of gyration of the column in inches. The maximum value of P is taken as 14,000 lb.

The American Bridge Company's Formula.—The American Bridge Company has adopted the following formula for the design of compression members.

Axial compression of gross sections of columns, for

ratio of l/r up to 120 .... $19{,}000 - 100l/r$
with a maximum of 13,000

where l = effective length of members in inches,
r = corresponding radius of gyration of section in inches.

For ratios of l/r up to 120, and for greater ratios up to 200, use the amounts given in the preceding table. For intermediate ratios, use proportional amounts.

A comparison of several compression formulas is given in Table IX.

TABLE IX.

Comparison of Compression Formulas.

Allowable Unit Stresses in Pounds per Square Inch.

American Bridge Company.

$\frac{l}{r}$	A. B. Co.	A. R. E. Ass'n. Chicago. Ketchum.	Gordon.	New York.	Philadelphia.	Boston.
		$16{,}000-70\frac{l}{r}$ 14,000 max.	$\frac{12{,}500}{1+\frac{l^2}{36{,}000\,r^2}}$.	$15{,}200-58\frac{l}{r}$.	$\frac{16{,}250}{1+\frac{l^2}{11{,}000\,r^2}}$.	$\frac{16{,}000}{1+\frac{l^2}{20{,}000\,r^2}}$.
0	13 000	14 000	12 500	15 200	16 250	16 000
5	13 000	14 000	12 490	14 910	16 215	15 980
10	13 000	14 000	12 460	14 620	16 100	15 920
15	13 000	14 000	12 420	14 330	15 925	15 820
20	13 000	14 000	12 365	14 040	15 680	15 690
25	13 000	14 000	12 285	13 750	15 375	15 515
30	13 000	13 900	12 195	13 460	15 020	15 310
35	13 000	13 550	12 090	13 170	14 620	15 075
40	13 000	13 200	11 970	12 880	14 185	14 815
45	13 000	12 850	11 835	12 590	13 725	14 530
50	13 000	12 500	11 690	12 300	13 240	14 220
55	13 000	12 150	11 530	12 010	12 745	13 900
60	13 000	11 800	11 365	11 720	12 240	13 560
65	12 500	11 450	11 185	11 430	11 740	13 210
70	12 000	11 100	11 000	11 140	11 240	12 850
75	11 500	10 750	10 810	10 850	10 750	12 490
80	11 000	10 400	10 615	10 560	10 275	12 120
85	10 500	10 050	10 410	10 270	9 810	11 755
90	10 000	9 700	10 205	9 980	9 360	11 390
95	9 500	9 350	9 995	9 690	8 930	11 025
100	9 000	9 000	9 785	9 400	8 510	10 670
105	8 500	8 650	9 570	9 110	8 115	10 315
110	8 000	8 300	9 355	8 820	7 740	9 970
115	7 500	7 950	9 140	8 530	7 380	9 630
120	7 000	7 600	8 930	8 240	7 035	9 300
125	6 750	7 250	8 715	......	6 715	......
130	6 500	6 900	8 510	......	6 405	......
135	6 250	6 550	8 300	......	6 115	......
140	6 000	6 200	8 095	......	5 840	......
145	5 750	5 850	7 890	......	......	......
150	5 500	5 500	7 690	......	......	......
155	5 250	......	7 495	......	......	......
160	5 000	......	7 305	......	......	......
165	4 750	......	7 120	......	......	......
170	4 500	......	6 935	......	......	......
175	4 250	......	6 755	......	......	......
180	4 000	......	6 580	......	......	......
185	3 750	......	6 410	......	......	......
190	3 500	......	6 240	......	......	......
195	3 250	......	6 080	......	......	......
200	3 000	......	5 920	......	......	......

TABLE IX.—*Continued.*

Name of Formula.	Abbreviation.	Maximum Ratio of l/r.	
		Main Members.	Bracing Struts.
American Bridge Company	A. B.	120	200
American Railway Engineering Association	A. R. E. A.	100	120
Chicago Building Law	C.	120	150
Ketchum's Specifications	K.	125	150
Gordon	G.	...	...
New York Building Law	N. Y.	120	...
Philadelphia Building Law	P.	140	...
Boston Building Law	B.	120	...

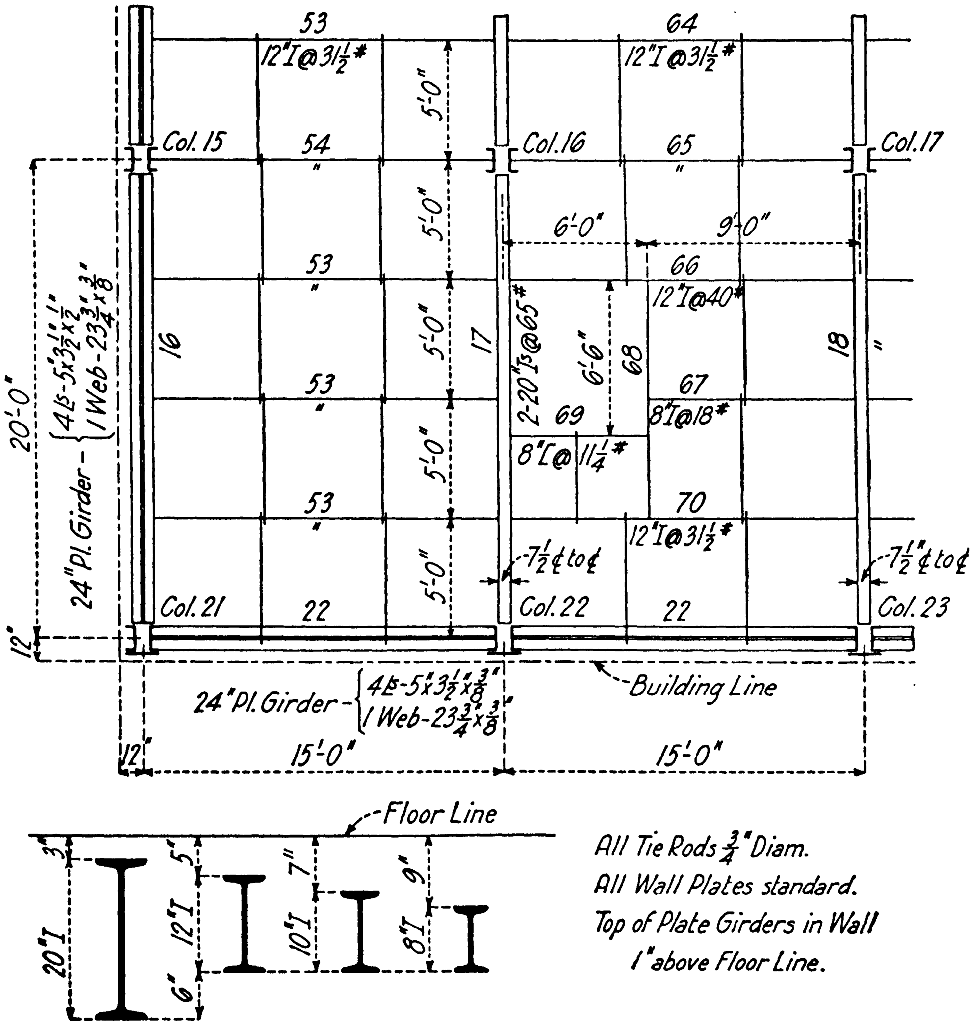
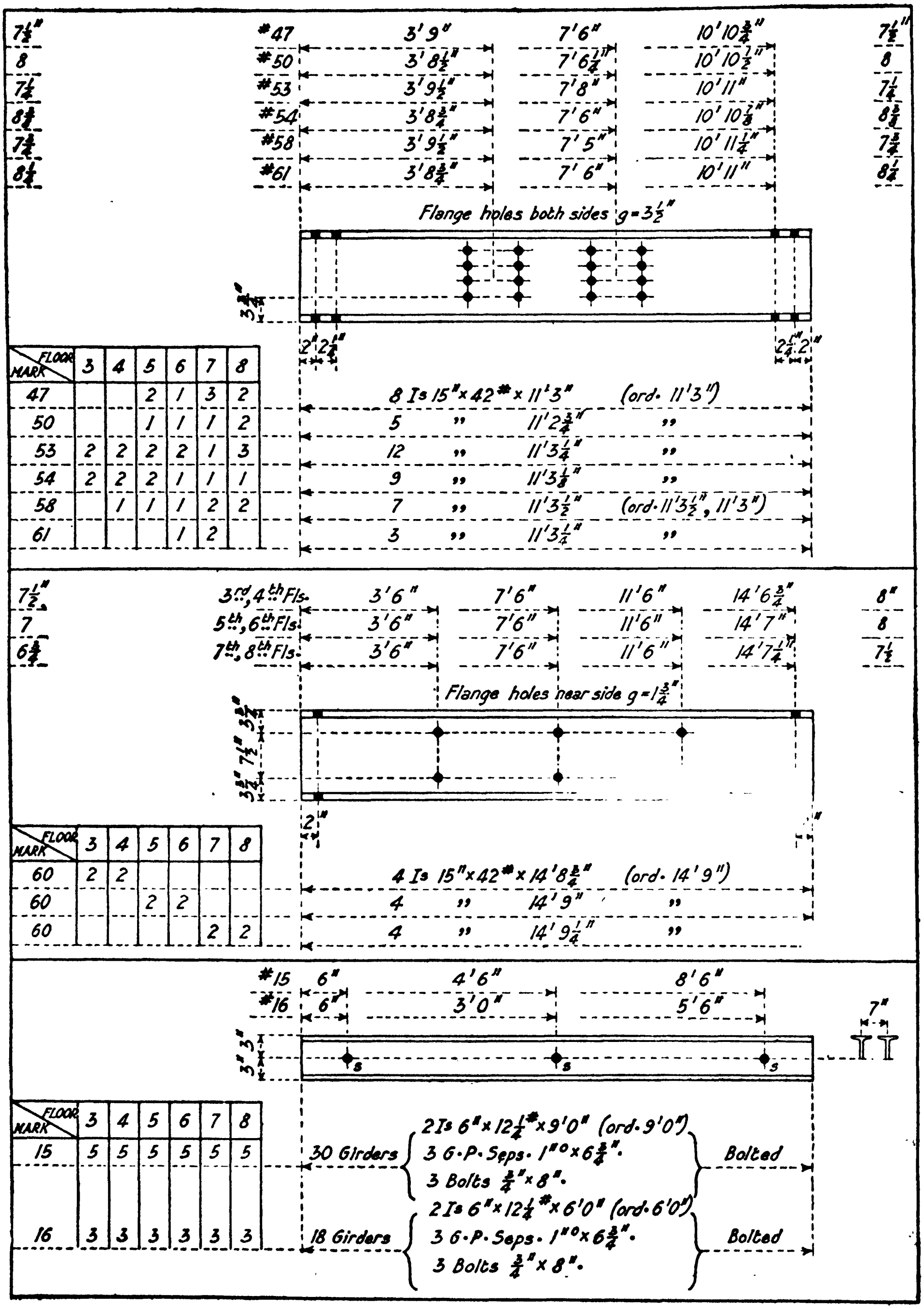


FIG. 2. FLOOR PLAN OF STEEL OFFICE BUILDING.



MARK \ FLOOR	3	4	5	6	7	8
47			2	1	3	2
50			1	1	1	2
53	2	2	2	2	1	3
54	2	2	2	1	1	1
58		1	1	1	2	2
61				1	2	

MARK \ FLOOR	3	4	5	6	7	8
60	2	2				
60			2	2		
60					2	2

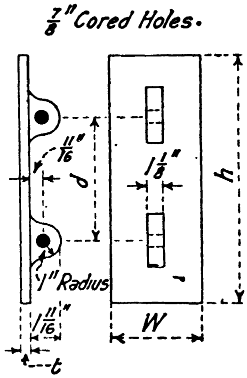
MARK \ FLOOR	3	4	5	6	7	8
15	5	5	5	5	5	5
16	3	3	3	3	3	3

FIG. 3. DETAILS OF FLOORBEAMS FOR A STEEL OFFICE BUILDING.

CAST IRON BEAM SEPARATORS

Beams				Separators						Bolts ¾"		
Size	Weight per Foot	Dist. c. to c. of Beams	Out to Out of Flange	w	h	d	t	Weight Each	Increase Weight For 1" Width	Length	Weight Including Nut	Increase Weight for 1" Length
24"	115#	8¾"	16¾"	8"	20"	12"	⅝"	31#	3·6#	10½"	3·6#	0·25#
	100	8	15½	7¼	20	12	⅝	28	3·6	10	3·5	0·25
	95 & 90	8	15¼	7¼	20	12	⅝	28	3·6	10	3·5	0·25
	85	8	15¼	7½	20	12	⅝	29	3·6	9½	3·3	0·25
	80	8	15	7½	20	12	⅝	29	3·6	9½	3·3	0·25
20"	100 & 95	8	15¼	7	16	12	⅝	22	2·9	10	3·5	0·25
	90	7½	14¾	6¾	16	12	⅝	22	2·9	9½	3·3	0·25
	85 & 80	7½	14½	6¾	16	12	⅝	22	2·9	9	3·2	0·25
20"	75	7½	14	6¾	16	12	⅝	22	2·9	9	3·2	0·25
	70	7	13½	6½	16	12	⅝	21	2·9	9	3·2	0·25
	65	7	13¼	6½	16	12	⅝	21	2·9	8½	3·2	0·25
18"	90	8	15¼	7	14	9	⅝	20	2·5	10	3·5	0·25
	85	8	15⅛	7¼	14	9	⅝	21	2·5	10	3·5	0·25
	80	8	15⅛	7¼	14	9	⅝	21	2·5	10	3·5	0·25
	75	8	15	7½	14	9	⅝	21	2·5	10	3·5	0·25
18"	70 & 65	7	13¼	6¼	14	9	⅝	18	2·5	9	3·2	0·25
	60	7	13¼	6½	14	9	⅝	19	2·5	8½	3·2	0·25
	55	7	13	6½	14	9	⅝	19	2·5	8½	3·2	0·25
15"	100 & 95	7½	14¼	6¼	11	7½	½	12	1·6	9½	3·3	0·25
	90	7½	14¼	6½	11	7½	½	12	1·6	9½	3·3	0·25
	85	7½	14	6½	11	7½	½	12	1·6	9½	3·3	0·25
15"	80 & 75	7	13¼	6	11	7½	½	12	1·6	9	3·2	0·25
	70 & 65	7	13¼	6¼	11	7½	½	12	1·6	9	3·2	0·25
	60	6½	12½	5¾	11	7½	½	11	1·6	8	3·0	0·25
15"	55	6½	12¼	5¾	11	7½	½	11	1·6	8	3·0	0·25
	50 & 45	6½	12¼	6	11	7½	½	12	1·6	8	3·0	0·25
	42	6½	12	6	11	7½	½	12	1·6	8	3·0	0·25
12"	55	6	11¾	5¼	8¾	5	½	9	1·3	8	3·0	0·25
	50	6	11½	5¼	8¾	5	½	9	1·3	8	3·0	0·25
12"	45	6	11¼	5¼	8¾	5	½	9	1·3	7½	2·9	0·25
	40 & 35	6	11¼	5½	8¾	5	½	9	1·3	7½	2·9	0·25
	31·5	6	11	5½	8¾	5	½	9	1·3	7½	2·9	0·25
10"	40	5½	10¾	4¾	7½		½	6	1·1	7½	1·4	0·13
	35	5½	10½	4¾	7½		½	6	1·1	7	1·4	0·13
	30	5½	10½	5	7½		½	7	1·1	7	1·4	0·13
	25	5½	10	5	7½		½	7	1·1	7	1·4	0·13
9"	35	5	10	4¼	6½		½	5	0·9	7	1·4	0·13
	30	5	9½	4¼	6½		½	5	0·9	6½	1·3	0·13
	25	5	9½	4½	6½		½	5	0·9	6½	1·3	0·13
	21	5	9¼	4½	6½		½	5	0·9	6½	1·3	0·13
8"	25·5	4½	9	4	5½		½	4	0·8	6	1·2	0·13
	23	4½	8¾	4	5½		½	4	0·8	6	1·2	0·13
	20·5 & 18	4½	8½	4	5½		½	4	0·8	6	1·2	0·13
7"	20	4½	8½	4	5		½	4	0·7	6	1·2	0·13
	17·5	4½	8¼	4	5		½	4	0·7	6	1·2	0·13
	15	4½	8¼	4¼	5		½	4	0·7	6	1·2	0·13
6"	17·25	4	7¾	3½	4½		½	4	0·6	5½	1·2	0·13
	14·75	4	7½	3½	4½		½	4	0·6	5½	1·2	0·13
	12·25	4	7½	3¾	4½		½	4	0·6	5½	1·2	0·13

For 5", 4" & 3" Beams use 1" gas pipe 3¼", 3" and 2¾" long respectively.



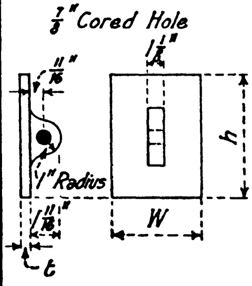


Fig. 4. Cast Iron Separators for Beams and Channels.
American Bridge Company.
(For details of separators for Bethlehem beams, see Part II.)

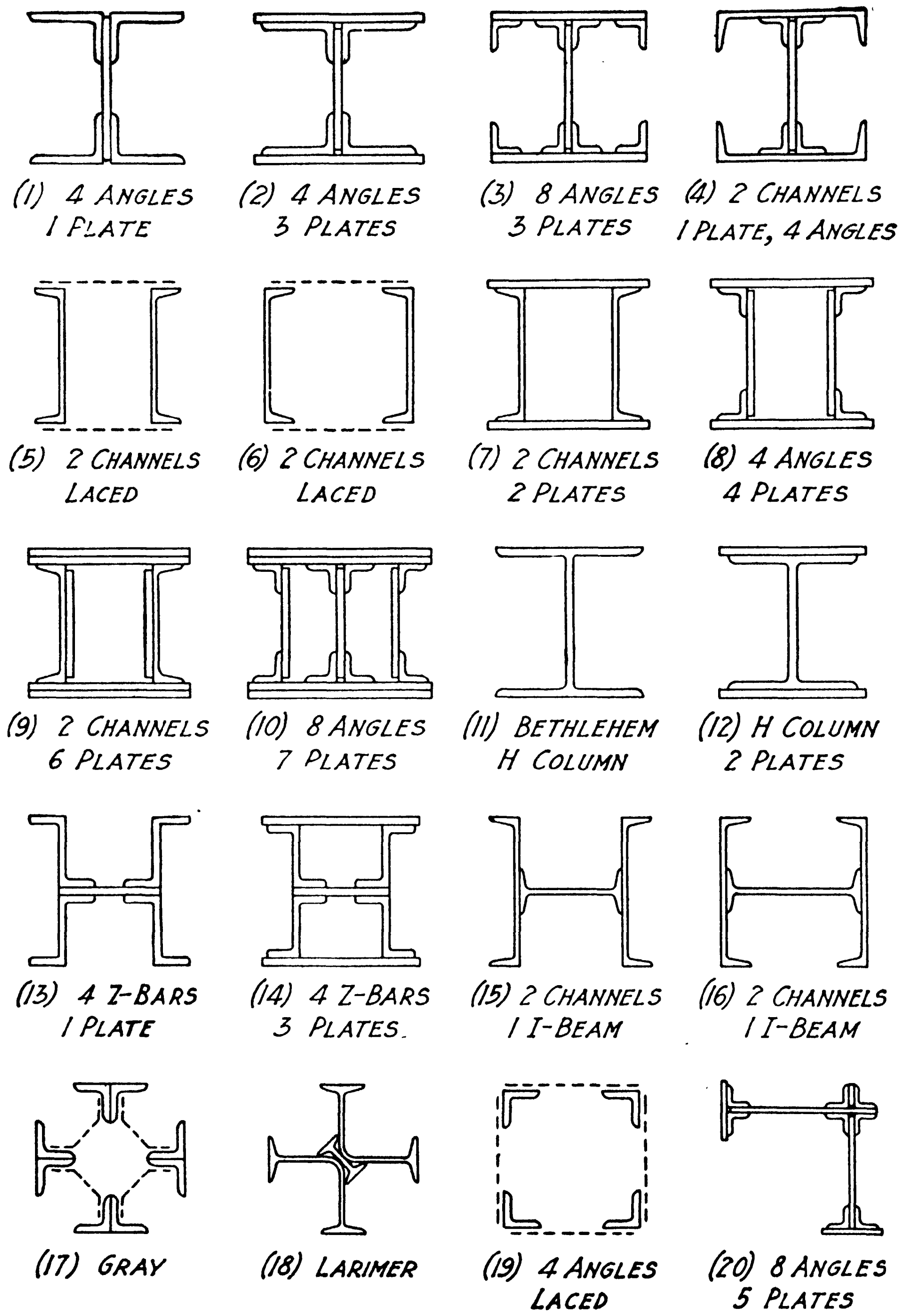


FIG. 5. TYPES OF COLUMNS FOR STEEL BUILDINGS.

DETAILS OF FRAMEWORK.—The framework of a steel skeleton building consists of floorbeams and floor girders which carry the floor loads to the columns, of columns which carry the loads to the foundations and of foundations which transfer the loads to the earth; the columns are braced transversely and longitudinally by wind bracing and by means of the floor girders, and the roof is carried on trusses or on roof beams or purlins. There is in addition miscellaneous framing to carry the outside walls and the cornice, and the framing around elevators, etc. For additional details, see Chapter XII, Structural Drafting.

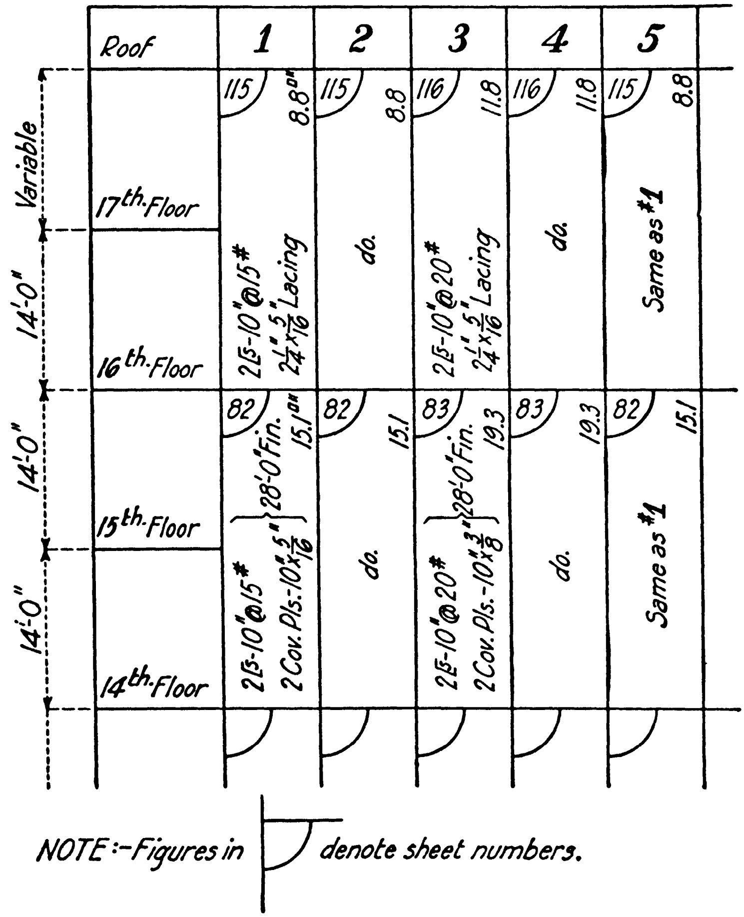


FIG. 6. COLUMN SCHEDULE.

Floor Plan.—The floor is carried on floorbeams to the floor girders and by the floor girders to the columns. A detail plan of a section of a floor plan of a steel skeleton building is shown in Fig. 2. The floorbeams, girders and columns are numbered as shown.

Details of floorbeams for an eight story steel office building are given in Fig. 3. For additional details of rolled beams and bracing, see Chapter XII. Details of cast separators are given in Fig. 4.

Columns.—Details of steel columns that are commonly used in steel skeleton buildings are given in Fig. 5. The built-H columns made of 4 angles and 1 plate or of 4 angles and 3 or 5 plates

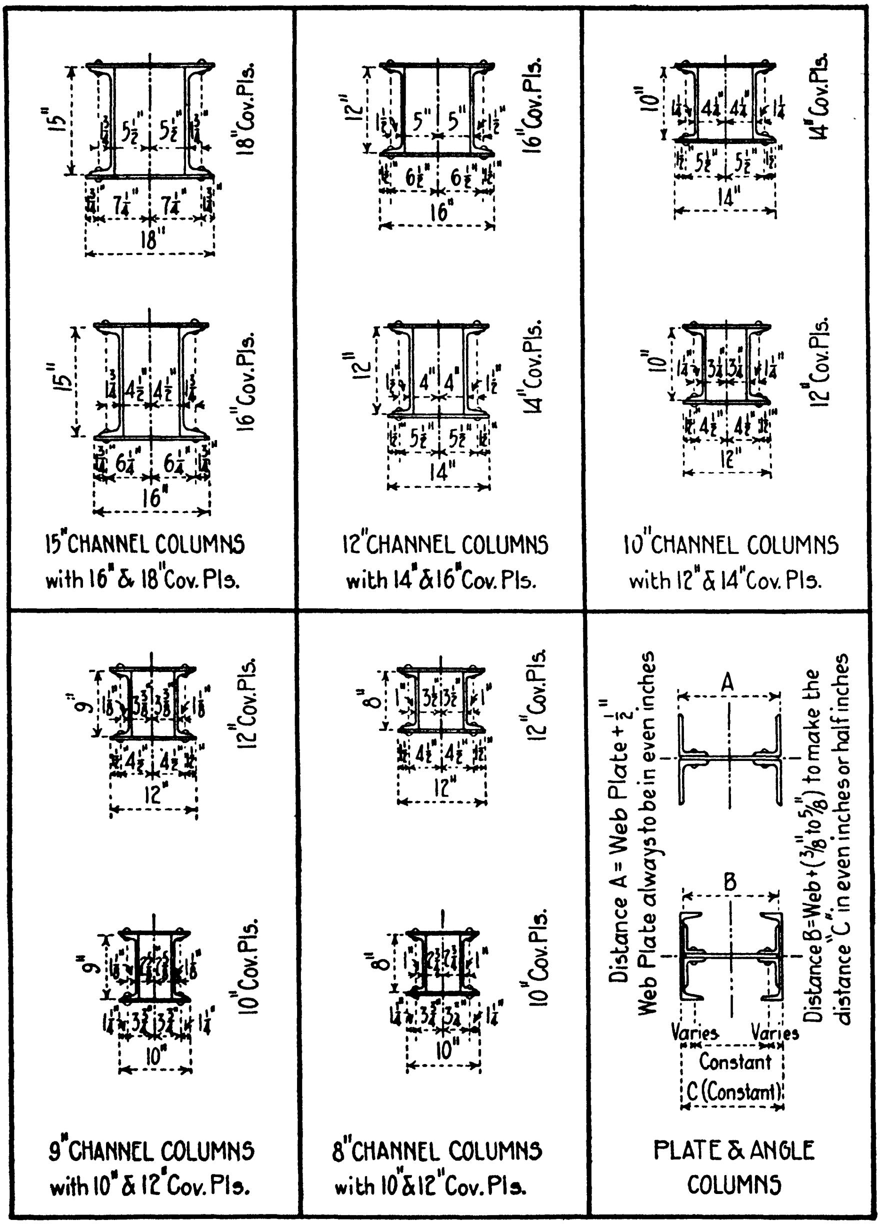


FIG. 7. DETAILS OF COLUMNS. AMERICAN BRIDGE COMPANY.

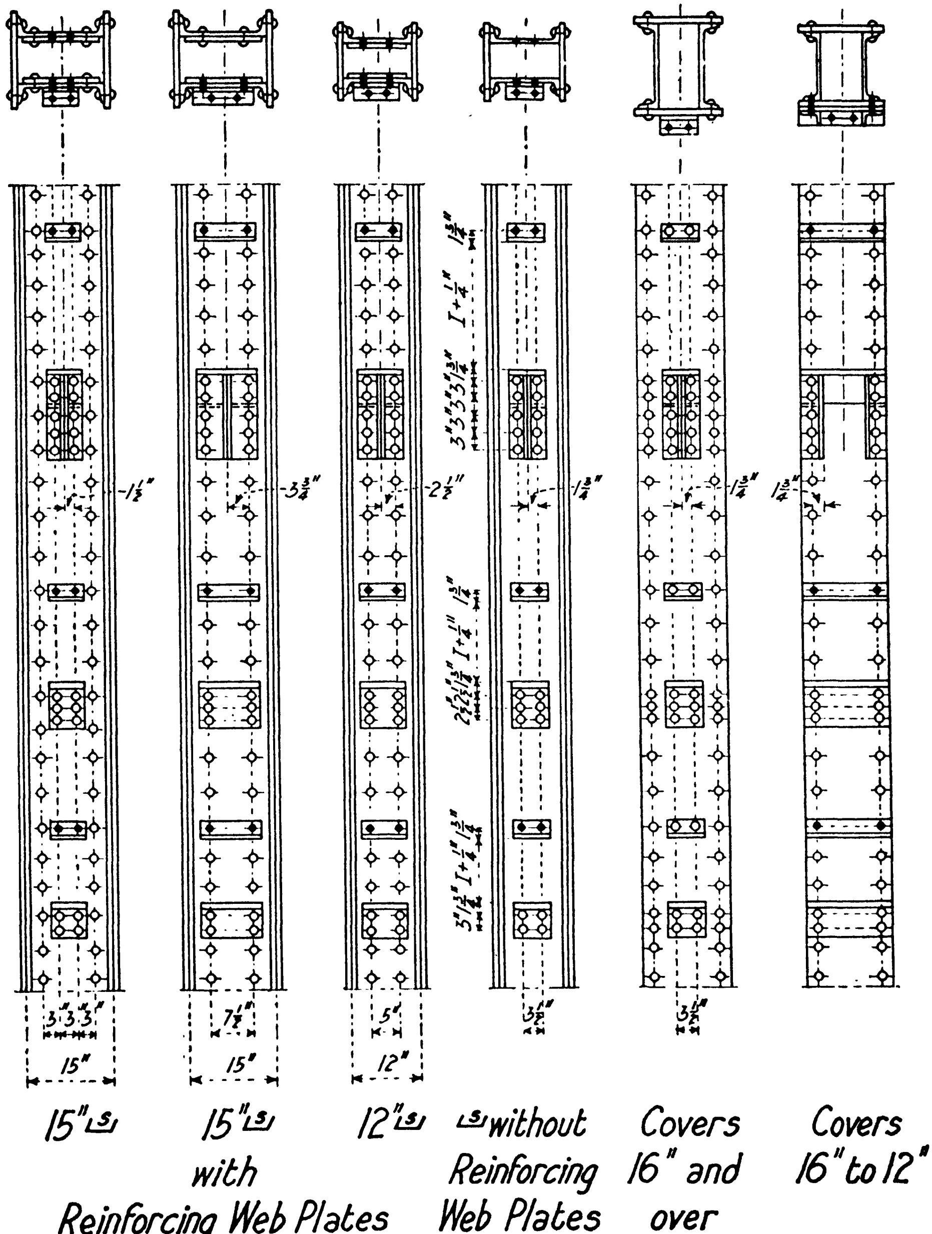


FIG. 8. DETAILS OF COLUMNS. AMERICAN BRIDGE COMPANY.

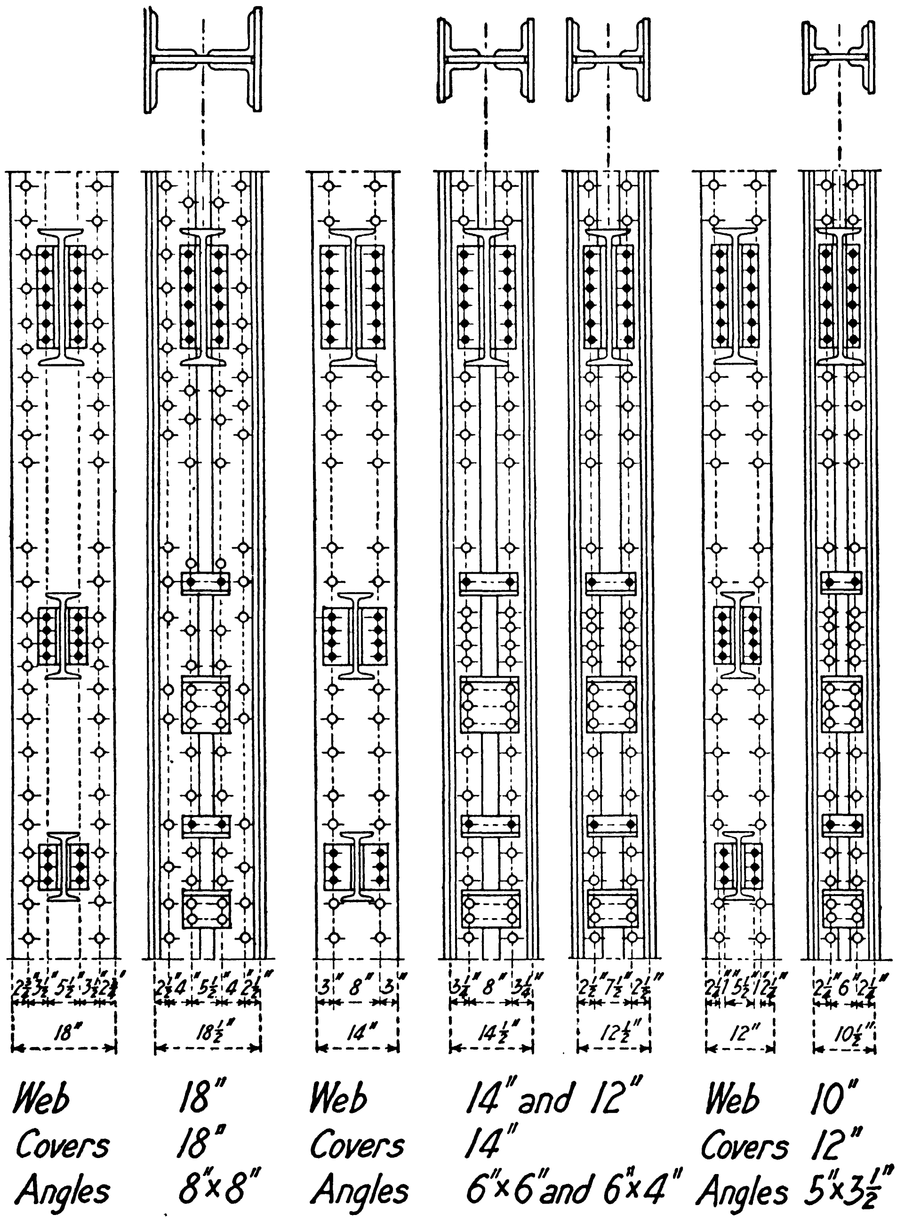


FIG. 9. DETAILS OF COLUMNS. AMERICAN BRIDGE COMPANY.

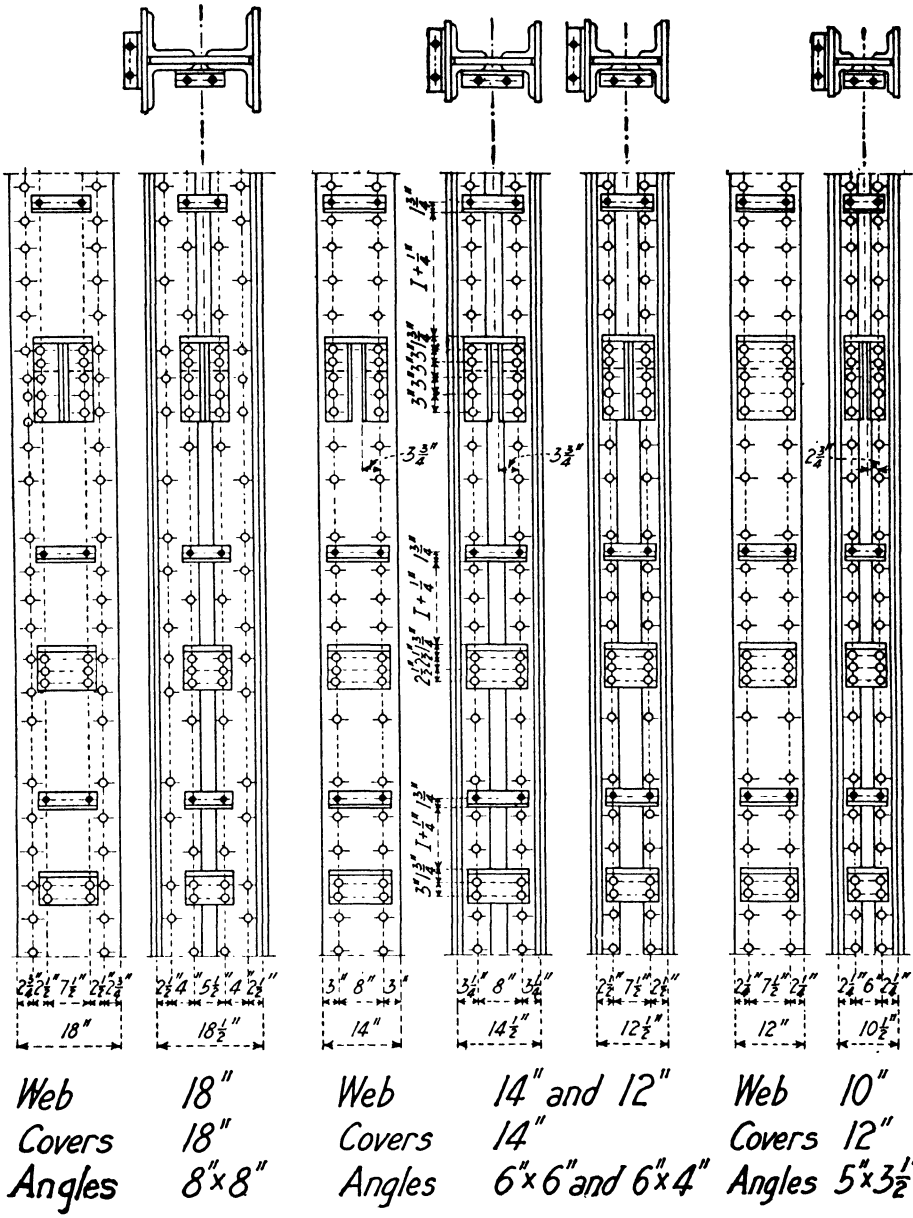


FIG. 10. DETAILS OF COLUMNS. AMERICAN BRIDGE COMPANY.

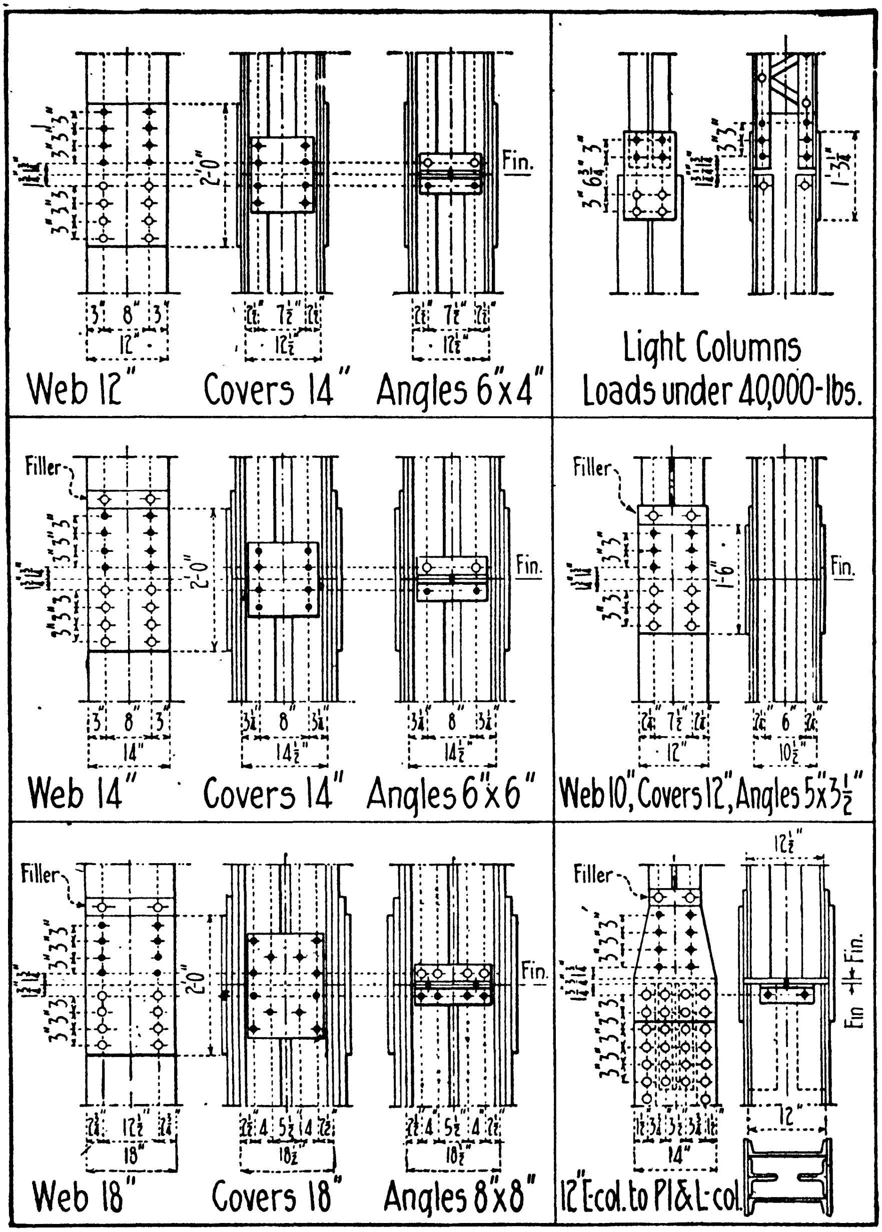


FIG. 11. DETAILS OF COLUMN SPLICES. AMERICAN BRIDGE COMPANY.

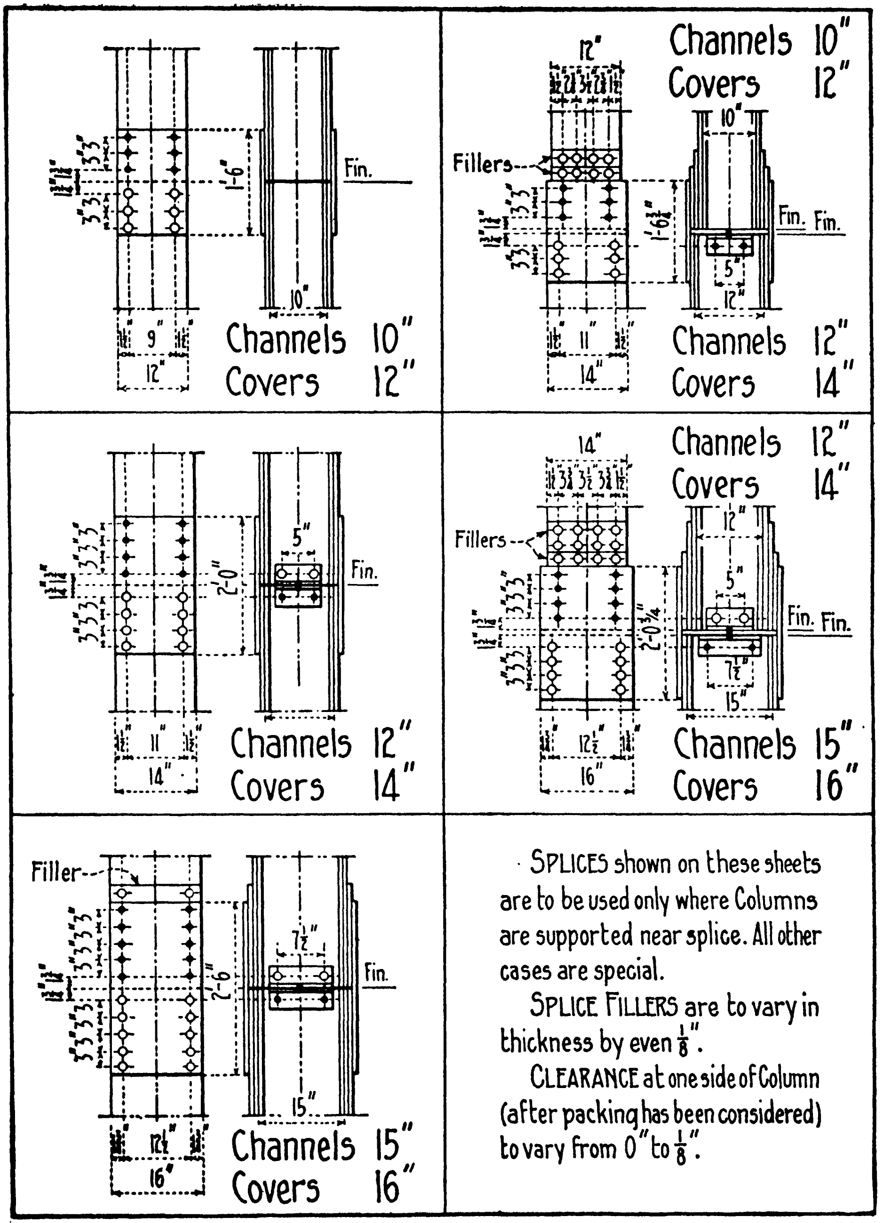
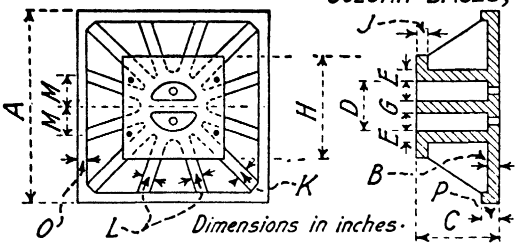


FIG 12. DETAILS OF COLUMN SPLICES. AMERICAN BRIDGE COMPANY.

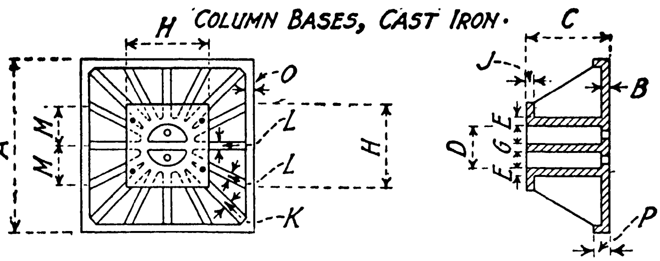
COLUMN BASES, CAST IRON



Drill holes in cap plate to suit column. Core two drain holes (one each side) $\frac{1}{2}$" diameter in lower end of hub. Core holes 2" diameter in base plate. Thickness of hub rib "G" applies only to bases used with channel columns. For plate & angle columns, hub rib "G" is to be increased when necessary.

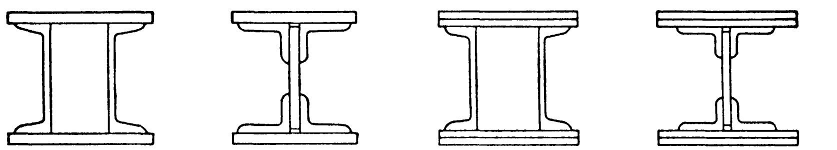
Base Plate		Height	Hub			Cap Plate		Ribs			Edge Rib		Estimated	Bearing Capacity		
A	B	C	Diam D	Thick E	Rib G	H	J	Cor. K	Int. L	Dist. M	O	P	Weight in lbs.	Thous lbs sq. ft.	Lbs per sq. in.	Total Thous Lbs.
2'0"	1"	9"	9"	1"	1"	1'6"	1"	1"	1"	4"	No Edge Rib	No Edge Rib	490	30	208	120
2-0	1 3/8	9	9	1	1	1-6	1	1	1	4			530	50	350	200
2-3	1 1/4	9	9	1	1	1-6	1	1	1	4 1/2			590	30	208	150
2-3	1 1/2	9	9	1	1	1-6	1 1/8	1	1	4 1/2			630	50	350	250
2-6	1 3/8	9	9	1	1	1-8	1	1	1	5			730	30	208	188
2-6	1 3/4	9	9	1 1/4	1 1/4	1-8	1 1/4	1	1	5			830	50	350	312
2-9	1 1/2	1-3	9	1 1/4	1 1/4	1-8	1 1/8	1	1	5 1/2			1 140	30	208	226
2-9	2	1-3	9	1 1/4	1 1/4	1-8	1 3/8	1	1	5 1/2			1 270	50	350	378
3-0	1 1/4	1-3	10	1 1/4	1 1/4	1-9	1 1/4	1	1	6	1"	2 1/2"	1 260	30	208	270
3-0	1 3/8	1-3	10	1 1/4	1 1/4	1-9	1 3/8	1 1/4	1 1/4	6	1	2 3/4	1 400	40	275	360
3-0	1 1/2	1-3	10	1 1/4	1 1/4	1-9	1 1/2	1 1/4	1 1/4	6	1 1/4	3	1 460	50	350	450
3-6	1 3/8	1-3	11	1 1/4	1 1/4	2-1	1 3/8	1 1/4	1 1/4	7	1	2 3/4	1 790	30	208	368
3-6	1 1/2	1-3	11	1 1/2	1 1/2	2-1	1 1/2	1 1/4	1 1/4	7	1 1/4	3	1 890	40	275	490
3-6	1 3/4	1-3	11	1 1/2	1 1/2	2-1	1 3/4	1 1/2	1 1/2	7	1 1/4	3 1/2	2 140	50	350	612
4-0	1 1/2	1-9	11	1 1/4	1 1/4	2-1	1 1/2	1 1/4	1 1/4	8	1 1/4	3	2 620	30	208	480
4-0	1 3/4	1-9	11	1 1/2	1 1/2	2-1	1 3/4	1 1/2	1 1/2	8	1 1/4	3 1/2	3 030	40	275	640
4-0	2	1-9	11	1 3/4	1 3/4	2-1	2	1 1/2	1 1/2	8	1 1/2	4	3 250	50	350	800
4-6	1 3/4	1-9	12	1 1/2	1 1/2	2-3	1 3/4	1 1/2	1 1/2	9	1 1/4	3 1/2	3 560	30	208	608
4-6	2	1-9	12	1 3/4	1 3/4	2-3	2	1 3/4	1 3/4	9	1 1/2	4	4 040	40	275	810
4-6	2 1/4	1-9	12	2	2	2-3	2 1/4	1 3/4	1 3/4	9	1 1/2	4 1/2	4 290	50	350	1012
4-9	1 3/4	1-9	13	1 1/2	1 1/2	2-5	1 3/4	1 1/2	1 1/2	9 1/2	1 1/4	3 1/2	3 880	30	208	676
4-9	2	1-9	13	1 3/4	1 3/4	2-5	2	1 3/4	1 3/4	9 1/2	1 1/2	4	4 400	40	275	902
4-9	2 1/4	1-9	13	2 1/4	2	2-5	2 1/4	1 3/4	1 3/4	9 1/2	1 1/2	4 1/2	4 720	50	350	1128

FIG. 13. CAST IRON COLUMN BASES. AMERICAN BRIDGE COMPANY.

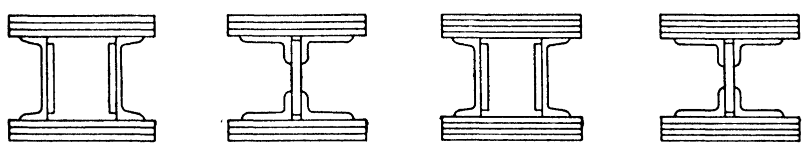


Base Plate		Height	Hub			Cap Plate		Ribs			Edge Rib		Estimated Weight in lbs.	Bearing Capacity		
A	B	C	Diam. D	Thick E	Rib G	H	J	Cor. K	Int. L	Dist. M	O	P		Thous lbs / sq. ft.	Lbs. per sq. in.	Total Thous. Lbs.
5′0″	$1\frac{3}{4}$″	2′3″	13″	$1\frac{3}{4}$″	$1\frac{1}{2}$″	2′5″	$1\frac{1}{2}$″	$1\frac{1}{2}$″	$1\frac{1}{2}$″	1′$0\frac{1}{2}$″	$1\frac{1}{4}$″	$3\frac{1}{2}$″	5 390	30	208	750
5-0	2	2-3	13	2	$1\frac{3}{4}$	2-5	$1\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{1}{2}$	1-$0\frac{1}{2}$	$1\frac{1}{2}$	4	5 850	40	275	1 000
5-0	$2\frac{1}{4}$	2-3	13	$2\frac{1}{4}$	2	2-5	2	2	$1\frac{3}{4}$	1-$0\frac{1}{2}$	$1\frac{1}{2}$	$4\frac{1}{2}$	6 550	50	350	1 250
5-6	$1\frac{3}{4}$	2-3	13	$1\frac{3}{4}$	$1\frac{1}{2}$	2-5	$1\frac{1}{2}$	$1\frac{3}{4}$	$1\frac{1}{2}$	1-$1\frac{3}{4}$	$1\frac{1}{4}$	$3\frac{1}{2}$	6 190	30	208	908
5-6	2	2-3	13	2	$1\frac{3}{4}$	2-5	$1\frac{3}{4}$	2	$1\frac{3}{4}$	1-$1\frac{3}{4}$	$1\frac{1}{2}$	4	7 010	40	275	1 210
5-6	$2\frac{1}{2}$	2-3	13	$2\frac{1}{4}$	2	2-5	2	$2\frac{1}{4}$	$1\frac{3}{4}$	1-$1\frac{3}{4}$	$1\frac{1}{2}$	5	7 780	50	350	1 512
6-0	2	2-9	13	2	$1\frac{1}{2}$	2-5	$1\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{1}{2}$	1-3	$1\frac{1}{2}$	4	8 250	30	208	1 080
6-0	$2\frac{1}{4}$	2-9	13	$2\frac{1}{4}$	$1\frac{3}{4}$	2-5	2	2	$1\frac{3}{4}$	1-3	$1\frac{1}{2}$	$4\frac{1}{2}$	9 280	40	275	1 440
6-0	$2\frac{1}{2}$	2-9	13	$2\frac{1}{2}$	2	2-5	2	$2\frac{1}{4}$	$1\frac{3}{4}$	1-3	$1\frac{1}{2}$	5	9 890	50	350	1 800

COLUMN SECTIONS



Channel Column One Cover Plate — Plate & Angle Column One Cover Plate — Channel Column Two Cover Plates — Plate & Angle Column Two Cover Plates



Channel Column Three Cover Plates — Plate & Angle Column Three Cover Plates — Channel Column Four Cover Plates — Plate & Angle Column Four Cover Plates

FIG. 14. STEEL COLUMN SECTIONS AND CAST IRON COLUMN BASES. AMERICAN BRIDGE COMPANY

as given in (1) and (2) are the most satisfactory columns for usual conditions. The Bethlehem H columns in (11) and (12) make very satisfactory columns. While the Bethlehem H columns require the driving of less rivets than are required to fabricate built-H columns, the extra cost required to drill from the solid in heavy Bethlehem H columns makes the final cost of the two types of columns practically the same for average conditions. Columns made of two channels laced are deficient in lateral rigidity and should only be used for light loads. Z-bars are difficult to obtain from the rolling mill and Z-bar columns should not be used unless it is known that Z-bars can be obtained. Additional sections are given in Fig. 14.

Column Schedule.—A column schedule should be prepared as in Fig. 6. The column schedule should give the length, area of cross-section and the composition of every column in the building. For the use of the shop draftsmen the dead load, wind load and eccentric stresses should be given for each column.

Column Details.—Standard details for channel columns and for plate and angle columns are given in Fig. 7. Details of channel columns are given in Fig. 8. Details of plate and angle columns are given in Fig. 9 and Fig. 10. Details of column splices are given in Fig. 11 and Fig. 12. Details of a column used in the Singer Building are shown in Fig. 27.

Column Bases.—Details of cast iron column bases as designed by the American Bridge Company are given in Fig. 13 and Fig. 14. Intermediate sizes may be obtained by interpolation.

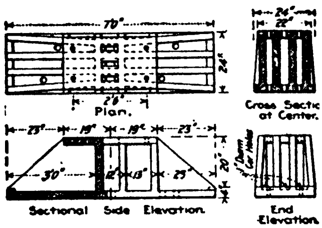


FIG. 15. CAST STEEL BASE.

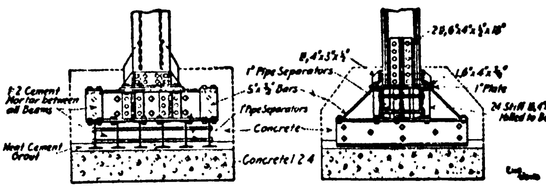


FIG. 16. BUILT STEEL COLUMN BASE.

Details of a cast steel column base used in the Singer Building are shown in Fig. 15. Details of a built steel column base designed by Mr. E. W. Stern, Consulting Engineer, are shown in Fig. 16 Mr. Stern considers the built steel column base as cheaper and more reliable than a cast steel base; and cheaper and very much more reliable than a cast iron base. In addition the base is easily set and readily grouted. After setting, the base is grouted with 1 to 2 Portland cement mortar. Bases of this design have been used for loads up to 1,600 tons.

Column bases are now (1924) made of steel slabs from 4 in. to 8 in. thick. See Eng. News-Record, April 27, 1922.

Anchors.—Details of anchors are given in Table 137, Part II. Anchors for columns in tall buildings should be calculated for the actual conditions.

Constant Dimension Columns.—The American Bridge Company has designed building columns which within certain limits for varying loads and areas, have constant overall dimensions. "Constant Dimension Columns" have the advantages (1) that their extreme dimensions are known in advance; (2) that wall columns can be spaced at a minimum distance from the outside limits of the building giving uniformity to the construction; (3) adjacent columns having different loads can have the same outside dimensions, thus reducing the number of column sizes in the building.

The typical sections of "Constant Dimension Columns" for several buildings are shown in Table 163, Part II. For high buildings where the lower columns are large the sizes can be reduced for the upper stories, but in general for office buildings, hotels and similar structures, after a

14-in. or a 16-in. column is reached, the size should be maintained with varying sections to the top of the building.

Changes in size of columns should be made on one floor or if this is not practicable certain groups should change at the same floor. No column should be smaller than 10 in., preferably not smaller than 12 in. Where framed connections and wind bracing are used care must be taken to select columns with sufficient clearance to permit the driving of long rivets. For the necessary clearance for driving field rivets see Table 164 to Table 170, Part II.

Dimensions, areas, radii of gyration and section modulii of " Constant Dimension Columns," as calculated by the American Bridge Company, are given in Table 162, Part II.

For office buildings of the usual type, about 23 stories high the selection of columns preferably should be either 16 in. columns throughout, or 16 in. for the first two lengths and 14 in. above. If wind moment or special loadings influence the selection, then 18-in. columns should be used for the first two lengths and 16 in. above.

For hotels about 15 stories high the selection of the columns preferably should be 14 in. throughout, or if special loading or wind stresses are involved, 16 in. throughout, or 16 in. for the first two lengths and 14 in. above.

For apartment houses of the usual type, about 12 stories high, 10 in. columns generally can be used throughout.

Steel Column Footings.—Column footings of rolled steel slabs and beam grillages, as designed by the American Bridge Company are given in Table 161, Part II. These footings are designed for a tensile stress of 16,000 lb. per sq. in., a shearing stress of 10,000 lb. per sq. in., and for the buckling of beam webs as specified by the Carnegie Steel Company in Table 9, Part II. In the tables T_1 = calculated thickness of the steel slab. The weights of footings include slabs, beams, separators and bolts. Footing numbers are descriptive and indicate the allowable pressure on the masonry foundation, the type of footing and the safe load in thousands of pounds.

Rolled steel slabs should be used in place of beam grillages where the required length of beam is 3 ft. or less. Single tier grillages should be used in preference to double grillages.

Slabs 4 in. thick or less may be straightened in the hydraulic press. Slabs over 4 in. thick should be planed where the surface bears on steel. Surfaces bearing on concrete need not be planed but the concrete should be grouted before setting the footing.

Gas pipe separators spaced one foot from the end of the beam and not more than 3 feet apart with ¾-in. bolts, should be provided in single tier grillages and in the upper tier of double grillages.

FOUNDATIONS.—The foundation for a tall building will depend upon the height of the structure, the total load on the foundation, the character of the soil, and the requirements of the design and may be briefly described as follows.

(1) Ordinary wall or pier foundations built on the natural soil.

(2) Walls and columns supported by timber grillage resting on the soil.

(3) Walls and columns supported on grillages made of steel beams or bars encased in concrete and resting on the soil.

(4) Piles of timber or concrete driven to rock or to a sufficient depth to carry the loads without settlement.

(5) Caissons as constructed in Chicago by excavating in an open well or shaft, curbing it with timber, and then filling the well with concrete.

(6) Caissons as constructed in New York by sinking steel cylinders, or steel and timber caissons, or reinforced concrete caissons, usually by the pneumatic process and filling the shaft with concrete. The first type of foundation, where the soil is compressible, can only be used for

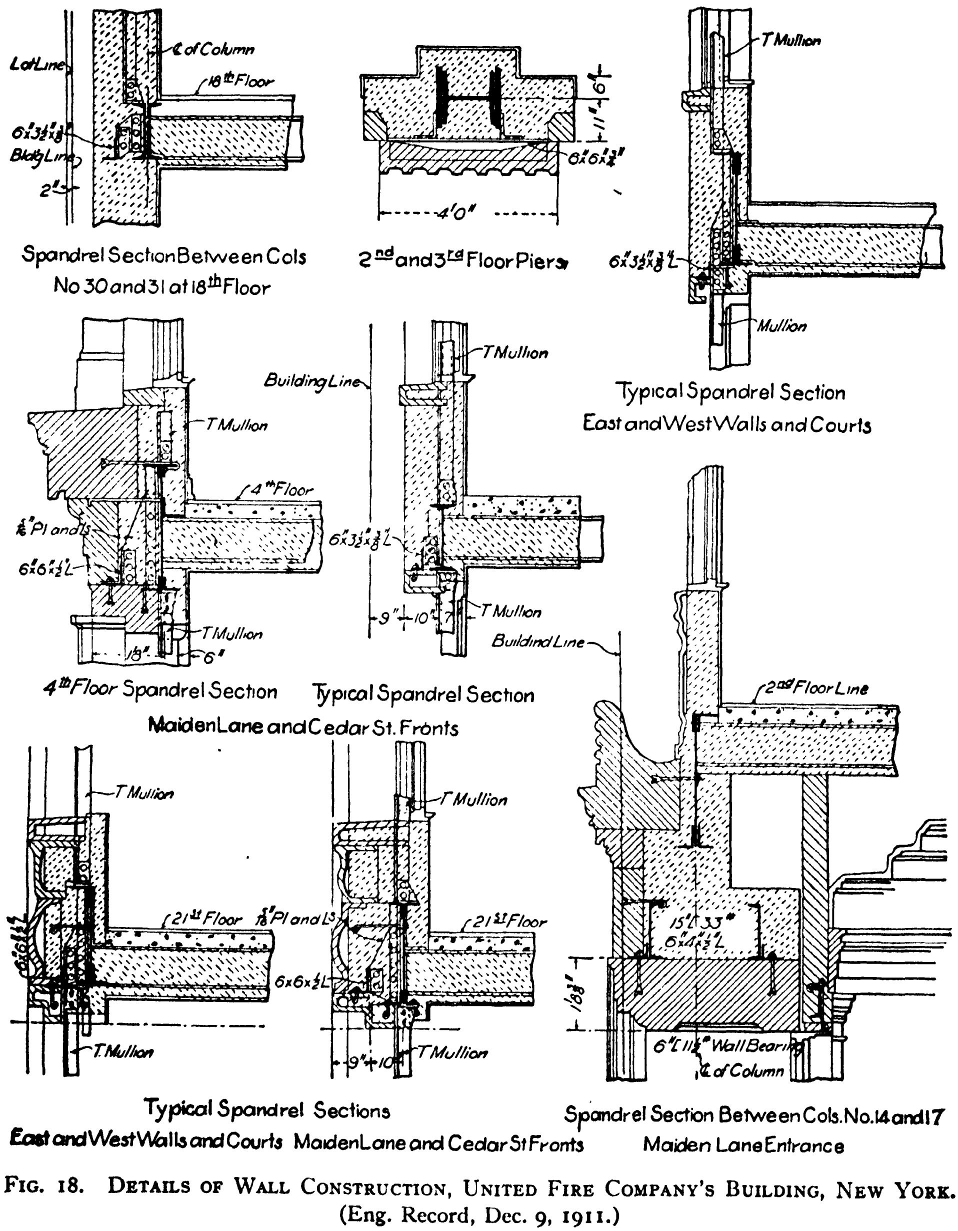


FIG. 18. DETAILS OF WALL CONSTRUCTION, UNITED FIRE COMPANY'S BUILDING, NEW YORK. (Eng. Record, Dec. 9, 1911.)

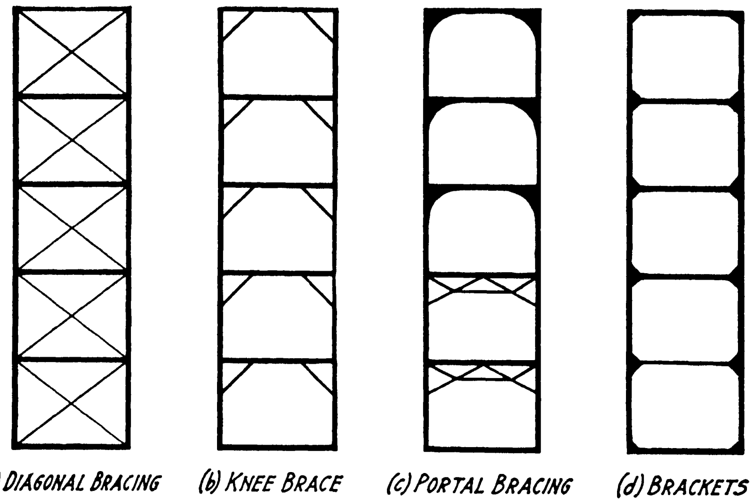


FIG. 19. TYPES OF WIND BRACING.

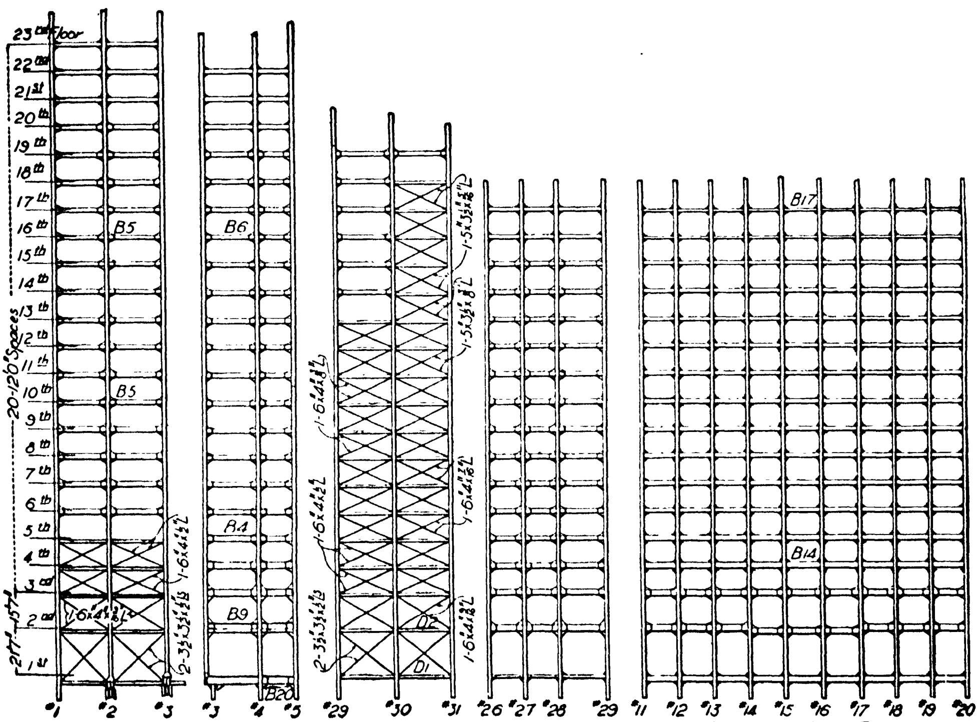


FIG. 20. WIND BRACING IN UNITED FIRE COMPANY'S BUILDING.
(Eng. Record, Dec. 9, 1911.)

buildings of four to six stories, but may be used for buildings of twelve to fifteen stories where the supporting power of the soil is considerable as in Denver. With high buildings the footings become so large as to be very expensive and also encroach upon the basement area.

Timber grillage and timber piles must be kept permanently wet or the life of the foundation will be very short. Many of the early tall buildings in Chicago were carried on timber grillages and on timber piles, but the settlement of the structures was so great that the method was abandoned for the method of concrete wells.

Steel grillage foundations have been much used for high buildings. With steel grillage the foundations may be made very shallow so that the basement is not encroached upon.

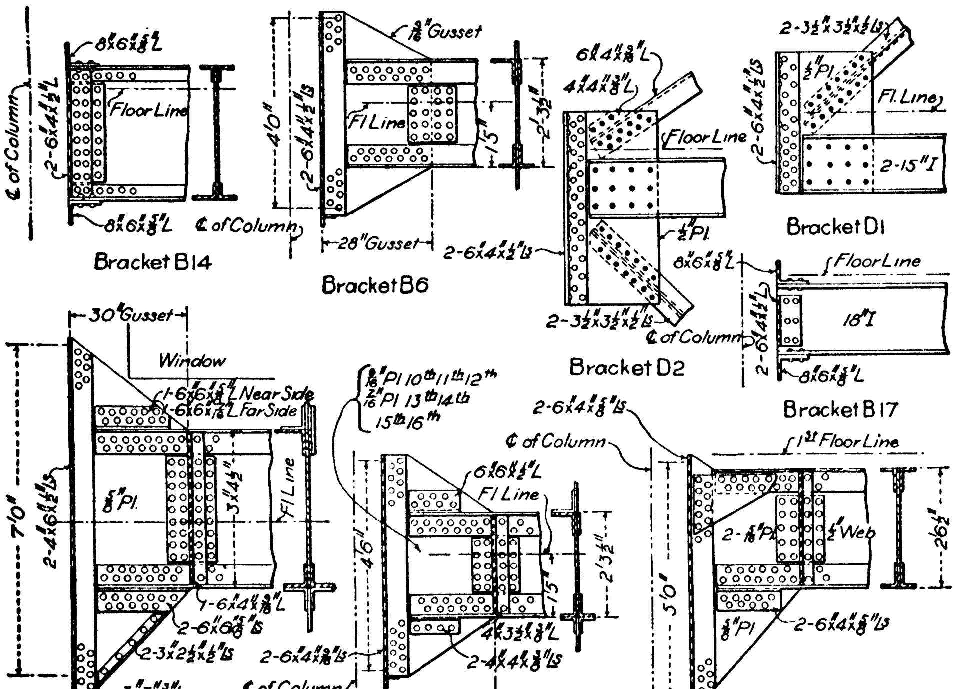


FIG. 21. DETAILS OF WIND BRACING IN UNITED FIRE COMPANY'S BUILDING. (Eng. Record, Dec. 9, 1911.)

In cities like Chicago and New York where real estate is so valuable that basements are often made three or four stories in depth, and where nearby disturbances due to excavations and tunneling would cause settlement it has been found necessary to carry the foundations to rock by means of wells or pneumatic caissons. In Chicago the wells commonly vary from 5 ft. to 12 ft. in diameter and are sunk in the open and are lined with timber curbing. After bed rock is reached the well is filled with concrete.

For a description of the sinking of the foundations for buildings in New York City, see a paper entitled "Foundations for the New Singer Building, New York City" by Mr. T. Kennard Thomson, Consulting Engineer, in Trans. Am. Soc. C. E., Vol. 63, June, 1909.

SPACING OF COLUMNS.—The spacing of columns in steel frame buildings varies from about 11 ft. to 24 ft., depending upon the height of the building, the floor loads, the type of floor

and other conditions. For buildings a few stories in height it is economical to space the columns closely together, while in high buildings a spacing of 16 ft. to 20 ft. will commonly be found economical. The columns in the Singer Tower in Fig. 22 were spaced 12 ft. centers; the columns in the Guaranty Trust Company's New York Building, 162 ft. high were spaced about 16 ft. by 16 ft. and 21 ft. 6 in. by 19 ft. 9 in.; the columns in the Woolworth Building, New York, were spaced at distances varying from 18 ft. 6 in. by 18 ft. 6 in. in the main part to a maximum of 28 ft. by 28 ft. in the tower.

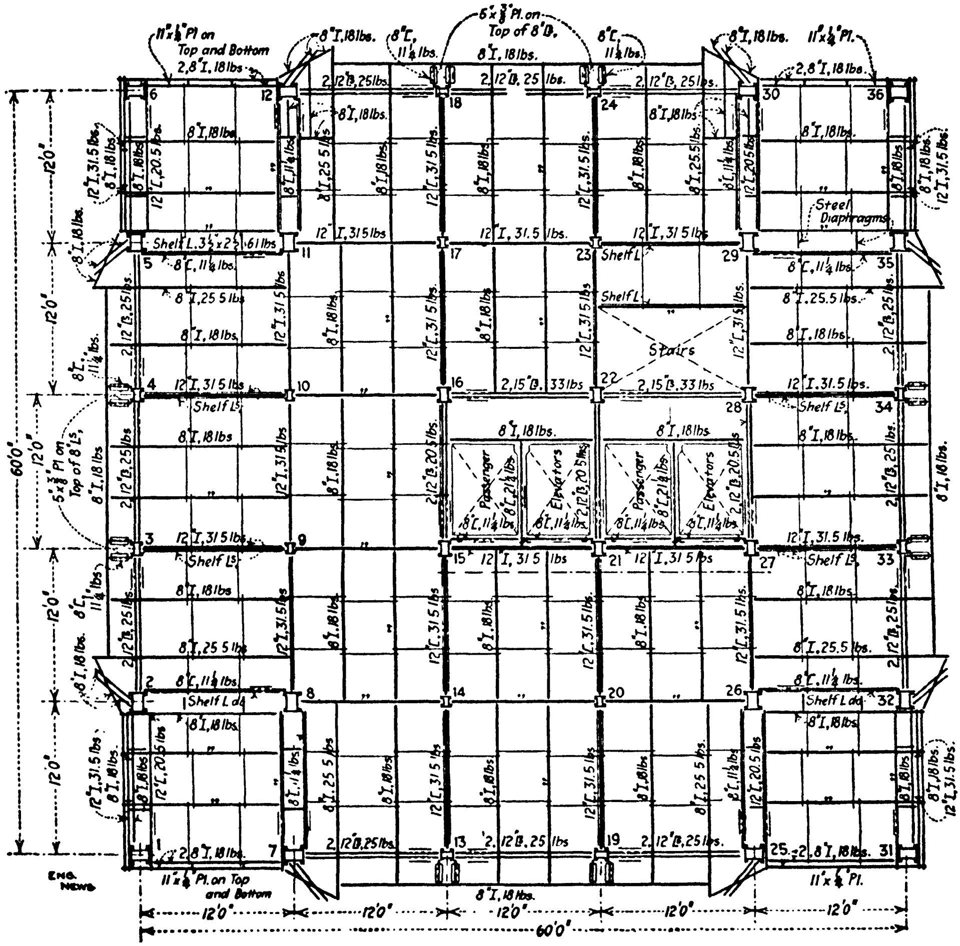


FIG. 22. TYPICAL FLOOR PLAN OF SINGER TOWER.

FLOOR PANELS.—For the long span system, floor girders connect the columns forming a square or rectangle, the floor slabs being supported on the floor girders. For the short span system, floorbeams are carried by the floor girders and the spans for the flooring are reduced. The spacing of the floorbeams will depend upon the type of floor, but it will commonly be found economical to use an even number of floorbeams giving an odd number of short spans in each panel. A common arrangement is to use two floorbeams which divide each panel into three short spans.

SPANDREL SECTIONS.—The design of the curtain walls that are supported by the spandrel beams will depend upon the material of which the wall is built, the amount and character of the ornamentation, and the details of the windows. The details of the wall construction in the United Fire Company's Building, New York, are given in Fig. 18. The spandrel masonry is carried by the wall girders and by horizontal angles bracketed from their outer faces. The angles in the outer flanges of the wall girders are often wider than those in the inner flanges to give additional support to the masonry, and both they and the detached spandrel angles have holes through their horizontal flanges to receive vertical expansion and wedge bolts to hold the stone or terra-cotta. The mullions over the windows are made of 3 in. by 4 in. tees.

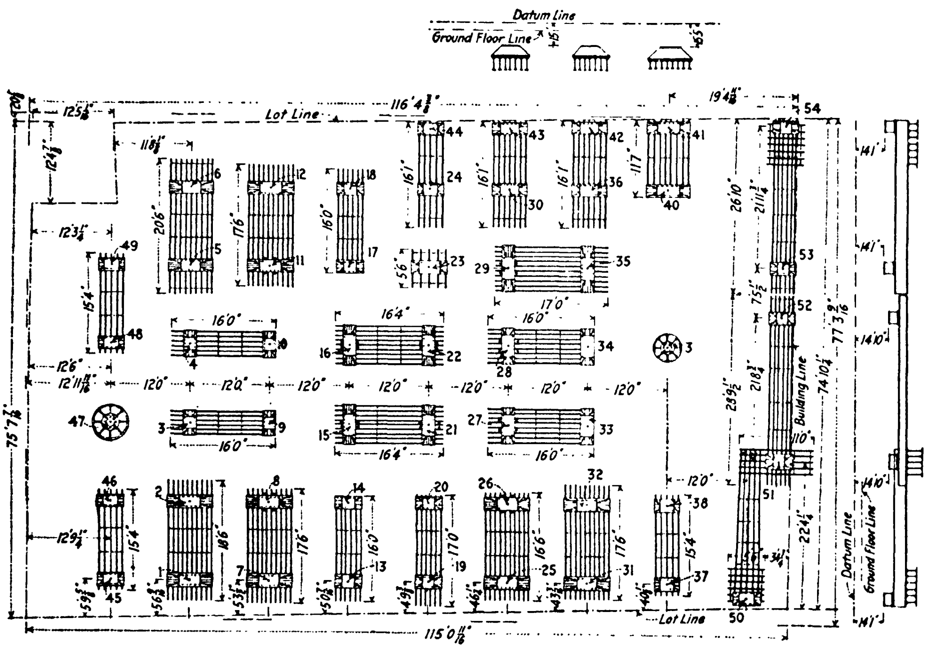


FIG. 23. FOUNDATION PLAN OF SINGER BUILDING.

The details of the spandrel walls should be worked out by the architect and the engineer working together if the best results are to be obtained.

WIND BRACING.*—The arrangement of the wind bracing in a steel frame building will depend upon the size and height of the building, upon the arrangement of the columns and the space that may be occupied by the wind bracing. Several types of wind bracing are shown in Fig. 19. Where space permits the diagonal bracing is the most effective. Diagonal bracing can only be used in solid walls or partitions. Knee braces (b) and portal bracing (c), can be used in outside walls where there is sufficient space above and below windows. Brackets (d) are used where the vertical clearance is limited and in wind bracing transversely through the building. Details of wind bracing of the United Fire Company's Building, New York, are given in Fig. 20 and Fig. 21. The building is 130 ft. 6 in. by 173 ft. 6 in. in plan and 25 stories in height. The columns are of Bethlehem H sections two stories in height. The floor panels are chiefly 15 ft. 6 in. by 24 ft. 3 in. The columns rest on grillages which rest on pneumatic piers.

Details of the wind bracing in the Singer Building are given in Fig. 24, Fig. 25, and Fig. 26.

* For the calculations of the stresses in eccentric riveted connections and in beams and girders transmitting wind movement, see Chapter XVII, and also Table 118a, Part II.

SINGER BUILDING.*—The Singer Building consists of a main portion approximately 75 ft. by 116 ft. in plan and 14 stories high, and a tower 60 ft. by 60 ft. in plan and 41 stories high with a four tier lantern which rises to a total height of 612 ft. The building is of skeleton steel con-

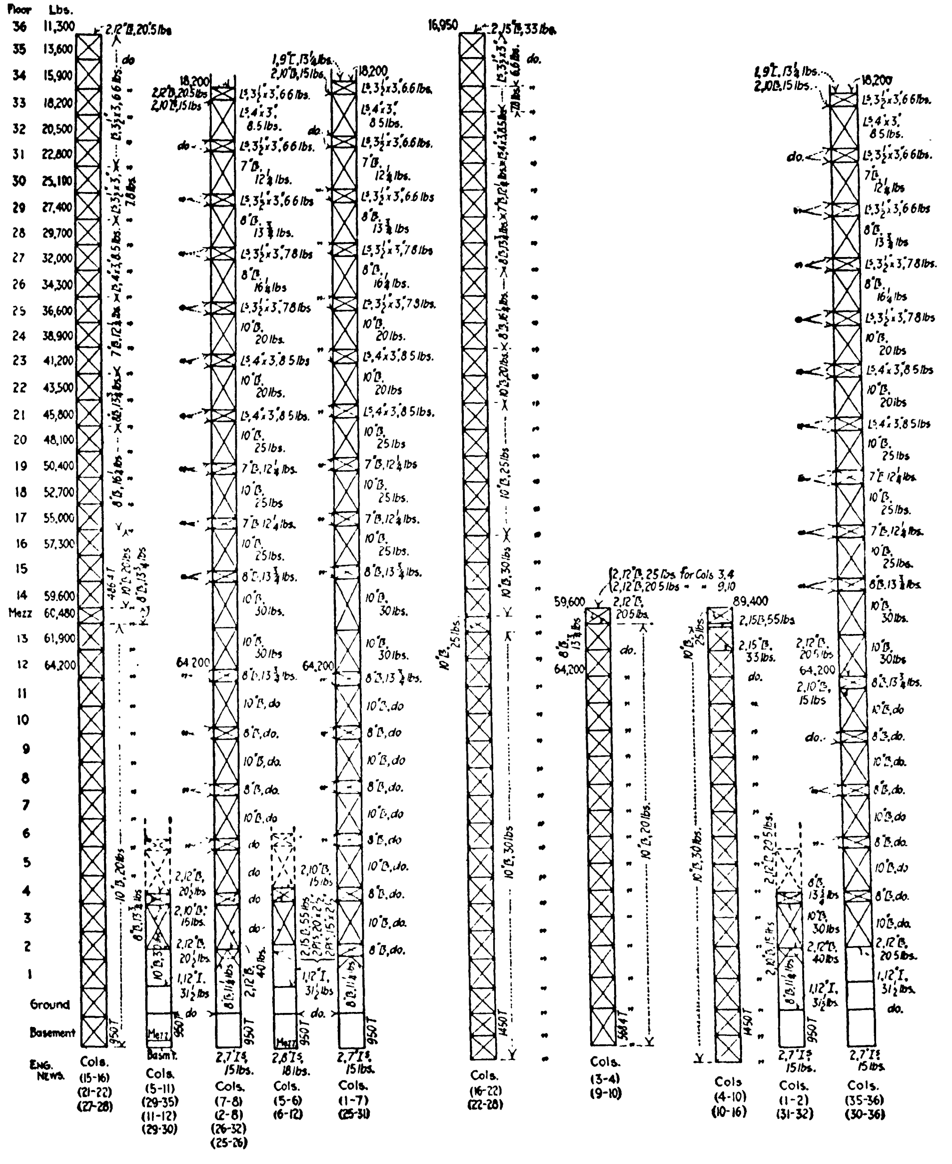


FIG. 24. DIAGRAM OF WIND BRACING, SINGER BUILDING.

struction, fireproofed with terra-cotta tiling and provided with terra-cotta floor systems surfaced with cement. The columns are carried on concrete footings sunk by the pneumatic process to a depth of 90 feet. The columns are spaced 12 ft. centers and are connected at right angles by

* Engineering News, Vol. 58, pp. 595 to 598.

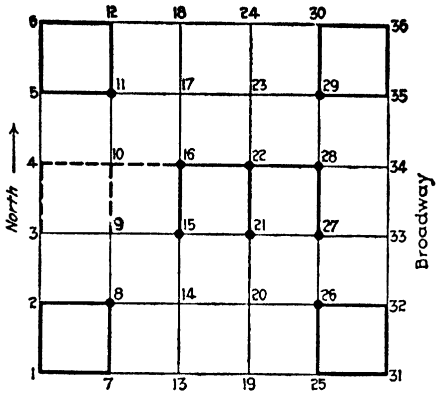


FIG. 25. PLAN OF WIND BRACING, SINGER BUILDING.

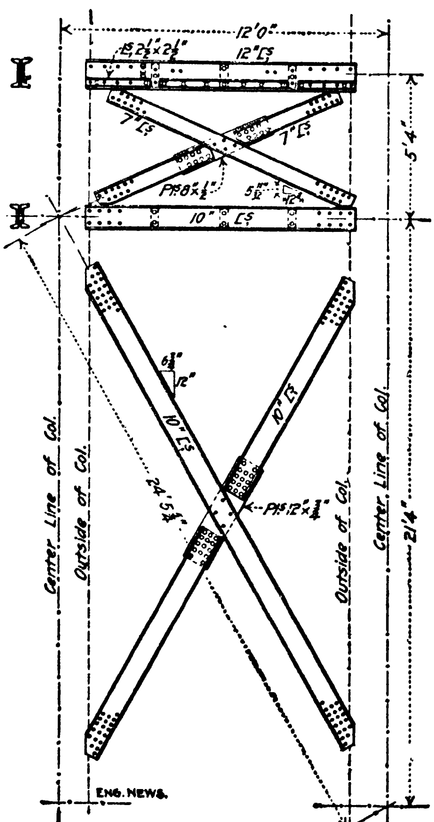


FIG. 26. DETAILS OF WIND BRACING, SINGER BUILDING.

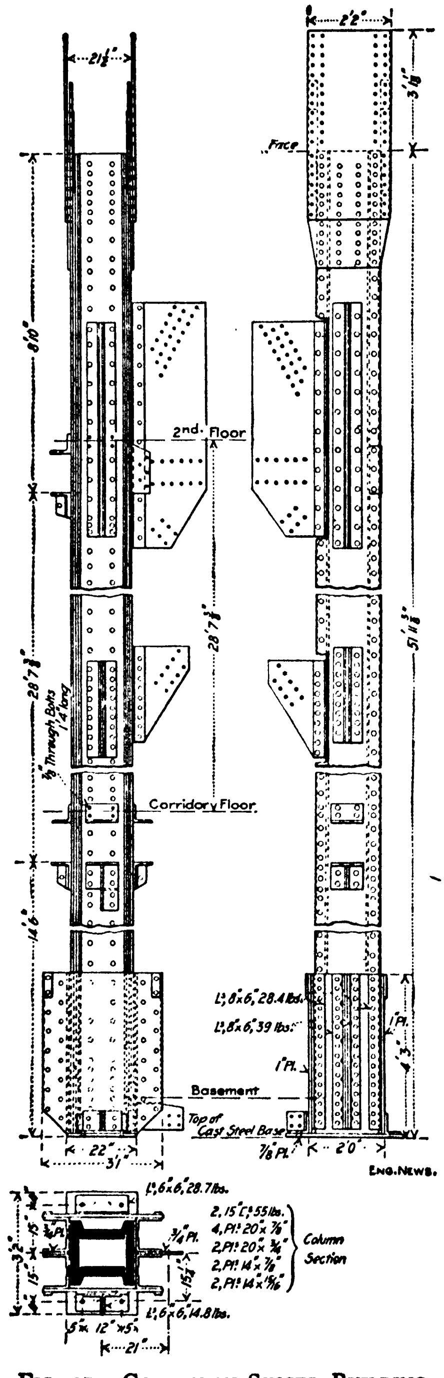


FIG. 27. COLUMN IN SINGER BUILDING.

girders and floorbeams. A typical floor plan of the tower is shown in Fig. 22. The columns are made of two channels, reinforced with plates where necessary. Details of a typical column are shown in Fig. 27. The wind bracing of the steel frame is shown in Fig. 24. A plan of the wind bracing in the tower is shown in Fig. 25. The panels that have heavy full lines were wind braced to the 33d story on the exterior and to the 36th story on the interior. Heavy dotted lines indicate wind bracing to the 14th story. Fine lines indicate no diagonal bracing. Circles on diagonal intersections represent anchor bolts. In designing the bracing the loads were distributed as follows:—It will be noticed that in a north and south direction there are 11 lines of wind bracing in the tower, nearly symmetrically placed. It was therefore assumed that on each story each line of X-bracing took $\frac{1}{11}$ of the total wind pressure of 30 lb. per sq. ft. The loads on the bracing in an east and west direction were distributed in a similar manner. The details of the X-bracing are shown in Fig. 26. Each of the 12 ft. square towers was assumed to act independently and the uplift of the columns was provided for.

SPECIFICATIONS FOR STEEL OFFICE BUILDINGS.

BY

MILO S. KETCHUM,

M. Am. Soc. C. E.

1924.

1. **Design.**—In all steel frame or skeleton buildings the stresses due to external and internal loads and wind stresses shall be transmitted to the foundation by the steel framework, no reliance being placed on the strength of the walls and partitions. Beams and girders shall have riveted connections to the steel columns. All columns shall be of structural steel with their different parts riveted together and shall be riveted to the beams and girders connecting to them.

2. **LOADS.**—The structure shall be designed to carry the following loads.

3. **Dead Loads.**—The dead load shall consist of the weight of all permanent construction and fixtures, such as walls, roofs, interior partitions, and fixed or permanent appliances. The weights of different materials shall be assumed as given in Table I. The minimum weight of fireproof floors to be assumed in designing the floor system shall be 75 lb. per sq. ft. The actual weight of floors shall be used in designing columns. The minimum weight of movable partitions shall be taken as 10 lb. per sq. ft.

4. **Live Loads.**—The live load shall consist of movable loads and loads due to machinery and other appliances.

The live loads required by Schneider's specifications and given in Table IV shall be used for the different classes of buildings. The maximum stresses due to any one of the three systems of loads shall be used in the design. Floor slabs for office buildings may be designed for a uniform load equal to twice the distributed load given in the second column of Table IV, and the effect of the concentrated load may be neglected. The concentrated load and load per linear foot of girder shall be considered in the design of all beams and girders. Flat roofs of office buildings, hotels, etc. that can be loaded by crowds of people shall be designed as the floors.

5. **Impact.**—For structures carrying traveling machinery such as cranes or conveyors, or machinery such as printing presses, 25 per cent shall be added to the stresses resulting from live load to provide for impact and vibrations.

6. **Snow Loads.**—The snow loads on roofs shall be taken the same as for steel frame mill buildings, Fig. 1, Chapter I.

7. **Wind Loads.**—All structures shall be designed to resist the horizontal wind pressure on the surface exposed above surrounding buildings as follows.

a. The wind pressure on roofs shall be taken as the normal component, calculated by Duchemin's formula, Fig. 3, Chapter I, of 30 lb. per square foot on the vertical projection of the roof.

b. The wind pressure on the sides and ends of buildings except as otherwise provided in the following paragraph shall be assumed as 20 lb. per square foot acting in any direction horizontally.

c. In designing the steel or reinforced concrete framework of fireproof buildings the framework shall be designed to resist a wind pressure of 30 lb. per square foot acting on the total exposed surface of all parts composing the framework or a horizontal wind pressure of 20 lb. per square foot acting in any direction horizontally on the sides and ends of the completed building. The strength of reinforced concrete floors may be considered in calculating the strength of the framework in the completed structure. The framework before the structure has been completed shall

be self-supporting without walls, partitions or floors. In no case shall the overturning moment due to wind pressure exceed 75 per cent of the resisting moment of the structure. In the calculations for wind bracing the working stresses for dead and live loads may be increased 25 per cent providing the sections are not less than required for dead and live loads. Chimneys shall be designed to resist a wind pressure of 20 lb. (⅔ of 30 lb.) per square foot acting on the vertical projection of the chimney. Curtain walls carried on the framework of steel or reinforced concrete buildings shall be designed to resist a horizontal pressure of 30 lb. per square foot acting horizontally on the outside of the entire surface of the wall.

8. **Minimum Loads on Roofs.**—Roofs shall be designed for the minimum loads specified by Schneider and given in Table VI.

9. **Live Loads on Columns.**—For columns carrying more than five floors, the live load may be reduced as follows:

For columns supporting the roof and top floor no reduction.

For columns supporting each successive floor a reduction of 5 per cent of the total live load may be made until 50 per cent is reached, which reduction of the load shall be used for the columns supporting all remaining floors. No column shall, however, be designed for a live load of less than 20,000 lb. The above reduction is not to apply to the live load on columns of warehouses, and similar buildings which are liable to be fully loaded on all floors at the same time.

10. **Loads on Foundations.** The loads on foundations shall not exceed the following in tons per square foot:

Ordinary clay and dry sand mixed with clay	2
Dry sand and dry clay	3
Hard clay and firm, coarse sand	4
Coarse sand and gravel	5
Shale rock	8
Hard rock	20

For all soils inferior to the above, such as loam, etc. never more than 1 ton per square foot.

The loads on foundations shall be assumed to be the same as for the footings of columns. The area of the bases of the foundation shall be proportioned for the dead load only as follows. That foundation which has the largest ratio of live load to dead load shall be selected and proportioned for the combined dead and live loads. The dead load on this foundation shall be divided by the area thus found, and this reduced pressure per square foot shall be the permissible pressure to be used for the dead loads of all foundations.

11. **Pressure on Masonry and Wall Plates.**—The maximum pressure on masonry and wall plates shall not be greater than the values given in Table VIII.

12. **Bases.**—Structural steel columns shall rest on either cast iron, cast steel or built steel bases proportioned so as to distribute entire load of the column on the concrete or masonry foundation. Columns carrying wind stresses shall be firmly anchored with at least two anchor bolts to a mass of concrete whose weight is at least 1½ times the up-lift in the column. All columns shall be properly secured to the bases.

13. **Shape of Foundations.**—Foundations under columns shall be symmetrical except under wall columns, where the center line of the column must lie within the middle third of the foundation. In this case the average intensity of the pressure on the soil shall not exceed one-half the safe load allowed for a symmetrical section. In cases where the wall column load exceeds the above safe loads the column must rest upon a steel or reinforced concrete girder or cantilever having a column or columns at the inner end. The foundation shall then be designed for the combined loads.

14. **Rolled Beams.**—The depth of rolled beams in floors shall be not less than one-twentieth of the span, and if used as roof purlins not less than one-thirtieth of the span. In case of floors subject to shocks and vibrations the depth of beams and girders shall be limited to one-fifteenth of the span. If shallower beams are used the sectional area shall be increased until the maximum deflection is not greater than that of a beam having a depth of one-fifteenth of the span, but the depth of such beams shall in no case be less than one-twentieth of the span.

15. **Expansion.**—Provision shall be made for expansion and contraction corresponding to a variation of temperature of 150 degrees Fahr. where necessary. Expansion rollers shall not be less than 4 inches in diameter.

16. **Cast Iron.**—The allowable stresses in cast iron shall be as follows:

Compression	= 12 000 lb. per sq. in.
Tension	= 2 500 lb. per sq. in.
Shear	= 1 500 lb. per sq. in.

17. **Steel Columns.**—Columns shall be of rolled or built sections. No wall column or column **with eccentric loads shall be used which does not have at least one solid plate or web of metal in or**

parallel to the plane of eccentric stress. Columns shall have a minimum length equal to two stories; and splices on adjacent columns shall preferably be made at different stories unless the building is symmetrical about a middle line of columns, in which case for ease in construction similarly situated columns may be made alike. Columns shall be designed so as to provide for effective connections for floorbeams, girders and brackets. The splices shall be strong enough to resist the bending stresses and make the columns practically continuous for their entire length. The splices of columns shall be riveted.

18. **Roof Trusses.**—Roof trusses shall be of steel and may have either pin or riveted connections, and shall be of such design that the stress in each member may be calculated. Roof trusses shall be braced in pairs and each pair of trusses shall be rigidly connected by lateral and transverse bracing. Purlins shall be made of shapes, or riveted plate or lattice girders. Trussed purlins will not be allowed. Main members of trusses shall be designed so that the neutral axes of intersecting members shall meet in a common point, or if this is not possible the eccentric stresses shall be calculated and provided for.

19. **Floorbeams.**—Floorbeams shall generally be rolled steel beams and shall be riveted to the floor girders by means of connection angles. Floor girders may be rolled beams or plate girders and shall be riveted to columns by means of connection angles. Shelf angles may be provided for convenience during erection.

The flange plates of all girders shall be limited in width so as not to extend beyond the outer line of rivets connecting them to the angles, more than 4 inches, or more than 8 times the thickness of the thinnest plate. For fireproof floors, floorbeams shall generally be tied together with tie rods at intervals not to exceed 8 times the depth of the beams. Tie rods are not required with reinforced concrete floors where the reinforcement is rigidly fastened to all outside beams and girders. Holes for tie rods, where the construction of the floor permits, shall be spaced 3 inches above the bottom of the beam.

Where more than one rolled beam is used to form a girder, they shall be connected by cast iron or steel separators and bolts spaced at intervals of not more than 5 feet. All beams having a depth of 12 inches and more shall have at least 2 bolts to each separator.

20. **Wall Plates.**—Bearing stones of granite, crystalline sandstone, or metal plates shall be used to reduce or distribute the pressure on the wall under the ends of wall beams, girders and trusses.

21. **Wall Anchors.**—The wall ends of beams, girders, and columns shall be anchored securely to give rigidity to the structure.

22. **Minimum Thickness of Metal.**—No plate or rolled section, having a thickness of less than $\frac{1}{4}$ in. shall be used except for fillers.

23. **Bracing.**—Lateral, longitudinal and transverse bracing shall preferably be composed of rigid members.

24. **Material.**—All parts of the structure shall be of rolled steel except column bases, bearing plates, separators or minor details which may be of cast iron or cast steel. The steel shall be made by the open-hearth process. All rolled steel, cast steel and cast iron shall comply with the "Specifications for Structural Steel for Buildings" adopted by the American Society for Testing Materials and printed in Chapter XV.

25. **Stresses.**—All parts of the structural framework shall be designed for the unit stresses as given in "Standard Specifications for Structural Steel for Buildings" by American Institute of Steel Construction, printed in last part of this chapter.

26. **Details of Construction.**—The details of construction shall comply with the specifications for steel frame buildings given in "Standard Specifications for Structural Steel for Buildings" by American Institute of Steel Construction, printed in last part of this chapter.

27. **Workmanship.**—The workmanship shall be equal to the best practice in modern bridge works and shall comply with the requirements in "Standard Specifications for Structural Steel for Buildings" by American Institute of Steel Construction, printed in last part of this chapter.

28. **Inspection and Testing at Mill and Shop.**—The specifications are the same as given in "Standard Specifications for Structural Steel for Buildings" by American Institute of Steel Construction, printed in last part of this chapter.

Erection.

29. **Tools.**—The contractor shall furnish at his expense all necessary tools, derricks, hoists, staging and material of every description required for the erection of the work, and shall remove same when the work is completed.

30. **Risks.**—The contractor shall assume all risks from storms or accidents, unless caused by the negligence of the owner, and all damage to adjoining property and to persons until the work is completed and accepted.

31. The contractor shall comply with all ordinances or regulations appertaining to the work.

32. **Details of Erection.**—The structural steel and iron work shall be erected as rapidly as the progress of the other work on the building will permit. Bases, bearing plates and ends of

girders which require to be grouted, shall be supported exactly at the proper level by means of steel wedges. Structural steel and ironwork shall be set accurately to the established lines and levels. The steel and iron must be plumb and level before riveting is commenced and must be kept in position until final completion. Temporary bracing shall be provided to resist the stresses due to derricks and other erection equipment. Elevator shafts shall be plumbed from top to bottom with piano wire. Riveted connections shall be carefully drawn up before riveting is commenced. Not less than one-third the holes shall be filled with field bolts, drawn up tight. All field connections shall be riveted. Pneumatic hammers shall be used in driving field rivets. Rivets must have a sufficient length to completely fill the holes and to form full heads. Rivets must be tight with full concentric heads. Loose or imperfect rivets must be cut out and redriven, recupping or calking will not be permitted. Holes which will not admit a cold rivet must be reamed. Where bolts are permitted, washers not less than ¼ in. thick shall be used under the nuts, the nuts shall be drawn tight and the threads checked with a chisel. Connections to cast iron and for separators in steel beams may be bolted.

STANDARD SPECIFICATIONS FOR STEEL BUILDINGS.—At the invitation of the American Institute of Steel Construction a committee consisting of George F. Swain, M. Am. Soc. C. E.; E. R. Graham of Graham, Anderson, Probst and White, Architects; W. J. Thomas, M. Am. Soc. C. E.; Wilbur J. Watson, M. Am. Soc. C. E., and Milo S. Ketchum, M. Am. Soc. C. E., prepared a "General Specification for Structural Steel for Buildings." This specification was transmitted to the American Institute of Steel Construction on June 1, 1923, with the following letter of transmittal.

"After careful deliberation the Committee selected to prepare a Standard Specification for the design, fabrication and erection of structural steel for buildings, submit the accompanying Code for your adoption.

"The present Specification contemplates that the inspection, is such that improper material containing defects which should cause rejection is not used. It is not intended to cover material salvaged from previous construction, which should not be used except under rigid supervision and inspection.

"It is also understood that the proper loads are taken and that impact is allowed for in each case by adding a proper percentage to the stresses produced by static live loads so that the total stress found in any member is an equivalent static stress. This Specification does not attempt to state definitely what the live, dead, or wind loads should be, or what percentage should be added for impact, as these are factors which should receive the careful consideration of competent engineers for each case. The question of corrosion under unusual conditions should have careful consideration by the engineer.

"The question of design is all-important. It necessarily presupposes that the design is good, made by and executed under the supervision of competent structural engineers; that proper provision is made for secondary stresses, excentric loads, unequal distribution of stresses on rivets, etc.; that the details are suitable and that the workmanship is high grade.

"It is recommended that the American Institute of Steel Construction maintain a Committee whose function shall be that of keeping such a Code as we submit consistent with the changing conditions of manufacture, design, and erection. Under these conditions, the Committee considers the unit stresses herein specified are proper."

STANDARD SPECIFICATION FOR STRUCTURAL STEEL FOR BUILDINGS,

As Adopted by the
American Institute of Steel Construction.

June 1, 1923.

1. This Specification defines the practice adopted by the American Institute of Steel Construction for the design, fabrication, and erection of structural steel for buildings.

2. **GENERAL.**—To obtain a satisfactory structure, the following major requirements must be fulfilled.

(a) The material used must be suitable, of uniform quality, and without defects affecting the strength or service of the structure.

(b) Proper loads and conditions must be assumed in the design.

(c) The unit stresses must be suitable for the material used.

(d) The workmanship must be good, so that defects or injuries are not produced in the manufacture.

(e) The computations and design must be properly made so that the unit stresses specified shall not be exceeded, and the structure and its details shall possess the requisite strength and rigidity.

3. **MATERIAL.**—Structural steel shall conform to the Standard Specifications of the American Society for Testing Materials for Structural Steel for Buildings, Serial Designation A 9–21, as amended to date.

4. **LOADING.**—

(a) Steel structures shall be designed to sustain the dead weight imposed upon them, including the weight of the steel frame itself, and, in addition, the maximum live load as specified in each particular case. Proper provision shall be made for temporary stresses caused by erection.

(b) In cases where live loads have the effect of producing impact or vibration, a proper percentage shall be added to the static live load stresses to provide for such influences, so that the total stress found in any member is an equivalent static stress.

(c) Proper provision shall be made for stresses caused by wind both during erection and after completion of the building. The wind pressure is dependent upon the conditions of exposure, but the allowable stresses specified in section five (5), paragraphs (f) and (g), are based upon the steel frame being designed to carry a wind pressure of not less than twenty (20) pounds per square foot on the vertical projection of exposed surfaces during erection, and fifteen (15) pounds per square foot on the vertical projection of the finished structure.

(d) Proper provision shall be made to securely fasten the reaction point of all steel construction and transmit the stresses to the foundations of the structures.

5. **ALLOWABLE STRESSES.**—All parts of the structure shall be so proportioned that the sum of the maximum static stresses in pounds per sq. in. shall not exceed the following:

(a) **Tension.**—Rolled Steel, on net section 18,000

(b) **Compression.**—Rolled Steel, on short lengths or where lateral deflection is prevented 18,000

On gross section of columns,

$$\frac{18{,}000}{1 + \dfrac{l^2}{18{,}000\, r^2}}$$

with a maximum of 15,000

In which l is the unsupported length of the column, and r is the corresponding least radius of gyration of the section, both in inches.

For main compression members, the ratio l/r shall not exceed 120, and for bracing and other secondary members, 200. (See Fig. 28.)

(c) **Bending.**—On extreme fibres of rolled shapes, and built up sections, net section, if lateral deflection is prevented 18,000

When the unsupported length l exceeds 15 times b, the width of the compression flange, the stress in pounds per sq. in. in the latter shall not exceed

$$\frac{20{,}000}{1 + \dfrac{l^2}{2{,}000\, b^2}}$$

The laterally unsupported length of beams and girders shall not exceed 40 times b the width of the compression flange. (See Fig. 29.)

On extreme fibres of pins, when the forces are assumed as acting at the center of gravity of the pieces 27,000

(d) **Shearing.**—

On pins 13,500

On power-driven rivets 13,500

On turned bolts in reamed holes with a clearance of not more than 1/50 of an inch ... 13,500

On hand-driven rivets 10,000

On unfinished bolts 10,000

On the gross area of the webs of beams and girders, where h, the height between flanges in inches, is not more than 60 times t, the thickness of the web in inches 12,000

On the gross area of the webs of beams and girders if the web is not stiffened where h, the height between flanges in inches, is more than 60 times t, the thickness of the web, the maximum shear per square inch, S/A shall not exceed

$$\frac{18{,}000}{1 + \dfrac{h^2}{7{,}200\, t^2}}$$

In which S is the total shear, and A is gross area of web in square inches. (See Fig. 30.)

(e) **Bearing.**—	Double Shear	Single Shear
On pins	30,000	24,000
On power-driven rivets	30,000	24,000
On turned bolts in reamed holes	30,000	24,000
On hand-driven rivets	20,000	16,000
On unfinished bolts	20,000	16,000

On expansion rollers per lineal inch 600 times the diameter of the roller in inches.

(f) **Combined Stresses.**—For combined stresses due to wind and other loads, the permissible working stress may be increased $33\frac{1}{3}$ per cent, provided the section thus found is not less than that required by the dead and live loads alone.

(g) **Members Carrying Wind Only.**—For members carrying wind stresses only, the permissible working stresses may be increased $33\frac{1}{3}$ per cent.

6. **Symmetrical Members.**—Sections shall preferably be symmetrical.

7. **Beams and Girders.**—

(a) **Rolled beams** shall be proportioned by the moment of inertia of their net section. Plate girders with webs fully spliced for tension and compression shall be so proportioned that the unit stress on the net section does not exceed the stresses specified in section five (5) as determined by the moment of inertia of the net section.

(b) **Plate girder webs** shall have a thickness of not less than 1-160 of the unsupported distance between the flanges.

(c) **Web splices** shall consist of a plate on each side of the web capable of transmitting the full stress through the splice rivets.

(d) **Stiffeners.**—Stiffeners shall be required on the webs of rolled beams and plate girders at the ends and at points of concentrated loads, and at other points where h the clear distance between flanges is greater than $85t\sqrt{18{,}000A/S-1}$, in which t is the thickness of the web. When stiffeners are required, the distance in inches between them shall not be greater than $85t\sqrt{18{,}000A/S-1}$, or not greater than 6 feet. When h is greater than 60 times t the thickness of the web of a plate girder, stiffeners shall be required at distances not greater than 6 feet apart. Stiffeners under or over concentrated loads shall be proportioned to distribute such loads into the web.

Plate girder stiffeners shall generally be in pairs, one on each side of the web, and shall have a close bearing against the flange angles at points of concentrated loading; stiffeners over the end bearings shall be on plate fillers. The pitch of rivet in stiffeners shall not exceed 6 in.

(e) **Flange plates** of all girders shall be limited in width so as not to extend more than 6 in. or more than 12 times the thickness of thinnest plate beyond the outer row of rivets connecting them to the angles.

(f) **Crane runway girders** and the supporting framework shall be proportioned to resist the greatest horizontal stresses caused by the operation of the cranes.

(g) **Rivets** connecting the flanges to the web at points of direct load on the flange between stiffeners shall be proportioned to carry the resultant of the longitudinal and tranverse shears.

(h) **Rivets** connecting the flanges to the webs of plate girders and of columns subjected to bending shall be so spaced as to carry the increment of the flange stress between the rivets.

8. **Column Bases.**—

(a) Proper provision shall be made to distribute the column loads on the footing and foundations.

(b) The top surface of all column bases shall be planed for the column bearing.

(c) Column bases shall be set true and level, with full bearing on the masonry, and be properly secured to the footings.

9. **Excentric Loading.**—Full provision shall be made for stresses caused by excentric loads.

10. **Combined Stresses.**—(a) Members subject to both direct and bending stresses shall be so proportioned that the greatest combined stresses shall not exceed the allowed limits.

(b) All members and their connections which are subject to stresses of both tension and compression due to the action of live loads shall be designed to sustain stress giving the largest section, with 50 per cent of the smaller stress added to it. If the reversal of stress is due to the action of wind, the member shall be designed for the stress giving the largest section and the connections proportioned for the largest stress.

11. **Abutting Joints.**—Compression members when faced for bearings shall be spliced sufficiently to hold the connecting members accurately in place. Other joints in riveted work, whether in tension or compression, shall be fully spliced.

12. **Net Sections.**—(a) In calculating tension members, the net section shall be used, and in deducting the rivet holes they shall be taken $\frac{1}{8}$ inch greater in diameter than the nominal diameter of the rivets.

(b) Pin-connected tension members shall have the section through the pin hole 25 per cent in excess of the net section of the member, and a net section back of the pin hole equal to 75 per cent of that required through the pin hole.

13. **Rivets and Bolts.**—(a) In proportioning rivets, the nominal diameter of the rivet shall be used.

(b) Rivets carrying calculated stresses, and whose grip exceeds five diameters, shall have their number increased 1 per cent for each additional 1/10 inch in the rivet grip. Special care shall be used in heating and driving such rivets.

(c) Rivets shall be used for the connections of main members carrying live loads which produce impact, and for connections subject to reversal of stresses.

(d) Finished bolts in reamed holes may be used in shop or field work where it is impracticable to obtain satisfactory power-driven rivets. The finished shank shall be long enough to provide full bearing, and washers used under the nuts to give full grip when turned tight.

Unfinished bolts may be used in shop or field work for connections in small structures used for shelters, and for secondary members of all structures such as purlins, girts, door and window framing, alignment bracing and secondary beams in floor.

14. **Rivet Spacing.**—(a) The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than $4\frac{1}{2}$ in. for $1\frac{1}{4}$ in. rivets, 4 in. for $1\frac{1}{8}$ in. rivets, $3\frac{1}{2}$ in. for 1 in. rivets, 3 in. for $\frac{7}{8}$ in. rivets, $2\frac{1}{2}$ in. for $\frac{3}{4}$ in. rivets, 2 in. for $\frac{5}{8}$ in. rivets, and $1\frac{3}{4}$ in. for $\frac{1}{2}$ in. rivets. The maximum pitch in the line of stress of compression members composed of plates and shapes shall not exceed 16 times the thinnest outside plate or shape, nor 20 times the thinnest enclosed plate or shape with a maximum of 12 in., and at right angles to the direction of stress the distance between lines of rivets shall not exceed 30 times the thinnest plate or shape. For angles in built sections with two gage lines, with rivets staggered, the maximum pitch in the line of stress in each gage line shall not exceed 24 times the thinnest plate with a maximum of 18 in.

(b) In tension members composed of two angles, a pitch of 3 ft. 6 in. will be allowed, and in compression members, 2 ft. 0 in., but the ratio l/r for each angle between rivets shall not be more than $\frac{2}{3}$ of that for the whole member.

(c) The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets for a length equal to $1\frac{1}{2}$ times the maximum width of the member.

(d) The minimum distance from the center of any rivet hole to a sheared edge shall be $2\frac{1}{4}$ in. for $1\frac{1}{4}$ in rivets, 2 in. for $1\frac{1}{8}$ in. rivets, $1\frac{3}{4}$ in. for 1 in. rivets, $1\frac{1}{2}$ in. for $\frac{7}{8}$ in. rivets, $1\frac{1}{4}$ in. for $\frac{3}{4}$ in. rivets, $1\frac{1}{8}$ in. for $\frac{5}{8}$ in. rivets, and 1 in. for $\frac{1}{2}$ in. rivets. The maximum distance from any edge shall be 12 times the thickness of the plate, but shall not exceed 6 in.

15. **Connections.**—(a) Connections carrying calculated stresses except for lacing, sag bars, or angles, hand rails, or beam connections, shall not have less than 2 rivets; or for field connections not less than 3 rivets.

(b) Members meeting at a joint shall have their lines of center of gravity meet at a point if practicable; if not, provision shall be made for any excentricity.

(c) The rivets at the ends of any member transmitting the stresses into that member should have their centers of gravity in the line of the center of gravity of the member; it not, provision shall be made for the effect of the resulting excentricity. Pins may be so placed as to counteract the effect of bending due to dead load.

(d) When a beam or girder " A " is connected to another member in such a manner that " A " acts as a continuous or fixed end beam, proper provision shall be made for the bending moments at such a connection.

(e) Where stress is transmitted from one piece to another, through a loose filler, the number of rivets shall be properly increased; tight-fitting fillers shall be preferred.

16. **Lattice.**—(a) The open sides of compression members shall be provided with lattice having tie plates at each end and at intermediate points if the lattice is interrupted. Tie plates shall be as near the ends as practicable. In main members carrying calculated stresses the end tie plates shall have a length of not less than the distance between the lines of rivets connecting them to the flanges, and intermediate ones of not less than one-half of this distance. The thickness of tie plates shall not be less than one-fiftieth of the distance between the lines of rivets connecting them to the segments of the members, and the rivet pitch shall not be more than four diameters. Tie plates shall be sufficient in size and number to equalize the stress in the parts of the members.

(b) Lattice bars shall have neatly finished ends. The thickness of lattice bars shall be not less than one-fortieth for single lattice and one-sixtieth for double lattice of the distance between end rivets; their minimum width shall be as follows:

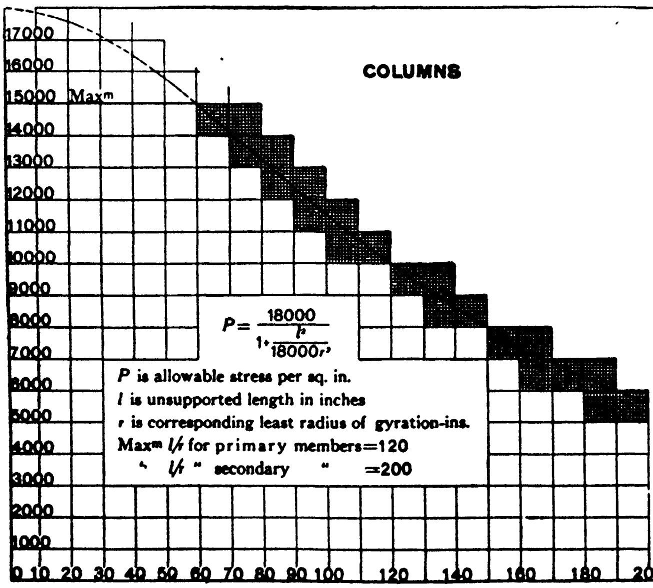
For 15 in. channels, or built sections with $3\frac{1}{2}$ in. and 4 in. angles—$2\frac{1}{4}$ in. ($\frac{7}{8}$ in. rivets), or $2\frac{1}{2}$ in. ($\frac{7}{8}$ in. rivets).

For 12 in., 10 in., and 9 in. channels, or built sections with 3 in. angles—$2\frac{1}{4}$ in. ($\frac{3}{4}$ in. rivets).

For 8 in. and 7 in. channels, or built sections with $2\frac{1}{2}$ in. angles—2 in. ($\frac{5}{8}$ in rivets), or $2\frac{1}{4}$ in. ($\frac{3}{4}$ in. rivets).

For 6 in. and 5 in. channels, or built sections with 2 in. angles—$1\frac{1}{2}$ in. ($\frac{1}{2}$ in. rivets), or $1\frac{3}{4}$ in. ($\frac{5}{8}$ in. rivets).

(c) The inclination of lattice bars to the axis of the members shall generally be not less than 45°; but when the distance between the rivet lines in the flanges is more than 15 inches, the lattice shall be double and riveted at the intersection if bars are used, or else shall be made of angles.



PRIMARY MEMBERS			
l/r	P	l/r	P
60	15000	90	12414
61	14916	91	12328
62	14832	92	12243
63	14748	93	12158
64	14661	94	12073
65	14578	95	11989
66	14492	96	11904
67	14407	97	11821
68	14321	98	11737
69	14235	99	11654
70	14150	100	11571
71	14062	101	11489
72	13975	102	11406
73	13888	103	11325
74	13801	104	11243
75	13714	105	11163
76	13627	106	11082
77	13540	107	11002
78	13452	108	10922
79	13366	109	10843
80	13279	110	10764
81	13192	111	10686
82	13104	112	10607
83	13018	113	10530
84	12931	114	10452
85	12844	115	10376
86	12757	116	10300
87	12672	117	10224
88	12585	118	10149
89	12500	119	10074
		120	10000

SECONDARY MEMBERS	
l/r	P
120	10000
125	9635
130	9283
135	8944
140	8617
145	8302
150	8000
155	7709
160	7431
165	7164
170	6908
175	6663
180	6429
185	6203
190	5989
195	5783
200	5586

FIG. 28. ALLOWABLE STRESSES IN COLUMNS. AMERICAN INSTITUTE OF STEEL CONSTRUCTION.

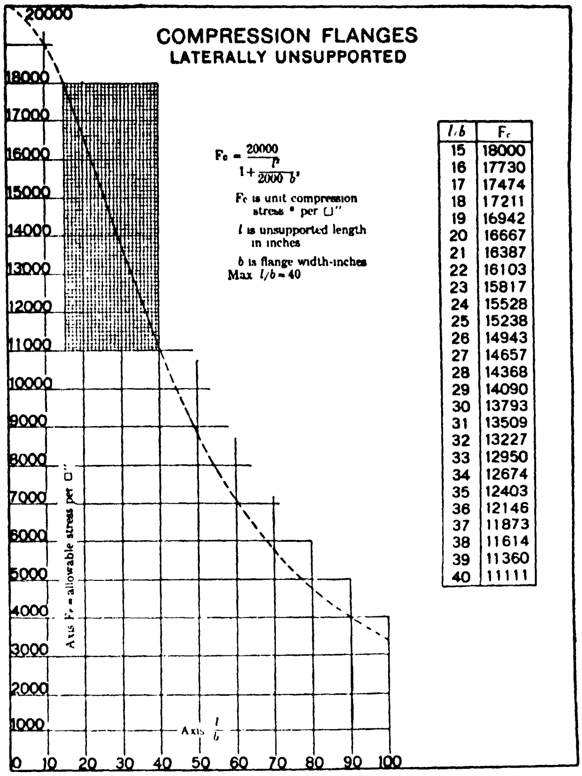


FIG. 29. ALLOWABLE STRESSES IN COMPRESSION FLANGES OF BEAMS AND GIRDERS. AMERICAN INSTITUTE OF STEEL CONSTRUCTION.

(d) Lattice bars shall be so spaced that the ratio l/r of the flange included between their connections shall be not over ¾ of that of the member as a whole.

17. **Expansion.**—Proper provision shall be made for expansion and contraction.

18. **Minimum Thickness.**—No steel less than 5/16 inch thick shall be used for exterior construction, nor less than ¼ inch for interior construction, except for linings or fillers and rolled structural shapes.

These provisions do not apply to light structures such as skylights, marquees, fire-escapes, light one-story buildings, or light miscellaneous steel work.

For trusses having end reactions of 35,000 lb. or over, the gusset plates shall be not less than ⅜ inch thick.

19. **Adjustable Members.**—The initial stress in adjustable members shall be assumed as not less than 5,000 lb.

20. **WORKMANSHIP.**—(a) All workmanship shall be equal to the best practice in modern structural shops.

(b) Drifting to enlarge unfair holes shall not be permitted.

(c) The several pieces forming built sections shall be straight and fit close together; and finished members shall be free from twists, bends, or open joints.

(d) Rolled sections, except for minor details, shall not be heated.

(e) Wherever steel castings are used, they shall be properly annealed.

(f) **Punching.**—Material may be punched 1/16 in. larger than the nominal diameter of the rivets, whenever the thickness of the metal is equal to or less than the diameter of the rivets, plus ⅛ in. When the metal is thicker than the diameter of the rivet, plus ⅛ in., the holes shall be drilled, or sub-punched and reamed.

(g) Rivets are to be driven hot, and wherever practicable, by power. Rivet heads shall

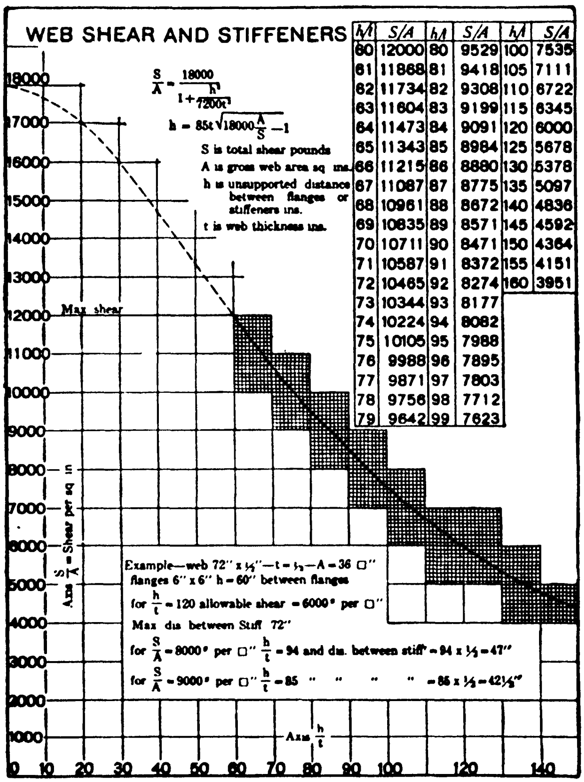
be of hemispherical shape and uniform size throughout the work for the same size rivet, full, neatly finished, and concentric with the holes. Rivets, after driving, shall be tight, completely filling the holes, and with heads in full contact with the surface.

(h) Compression joints depending upon contact bearing shall have the bearing surfaces truly faced after the members are riveted. All other joints shall be cut or dressed true and straight, especially where exposed to view.

(i) The use of a burning torch is permissible if the burned metal is not carrying stresses during the burning. Stresses shall not be transmitted into the metal through a burned surface.

21. **Painting.**—(a) Parts not in contact, but inaccessible after assembling, shall be properly protected by paint.

(b) All steel work, except where encased in concrete, shall be thoroughly cleaned and given one coat of acceptable metal protection well worked into the joints and open spaces.



h/t	S/A	h/t	S/A	h/t	S/A
60	12000	80	9529	100	7535
61	11868	81	9418	105	7111
62	11734	82	9308	110	6722
63	11604	83	9199	115	6345
64	11473	84	9091	120	6000
65	11343	85	8984	125	5678
66	11215	86	8880	130	5378
67	11087	87	8775	135	5097
68	10961	88	8672	140	4836
69	10835	89	8571	145	4592
70	10711	90	8471	150	4364
71	10587	91	8372	155	4151
72	10465	92	8274	160	3951
73	10344	93	8177		
74	10224	94	8082		
75	10105	95	7988		
76	9988	96	7895		
77	9871	97	7803		
78	9756	98	7712		
79	9642	99	7623		

FIG. 30. ALLOWABLE SHEAR IN WEBS OF GIRDERS. AMERICAN INSTITUTE OF STEEL CONSTRUCTION.

(c) Machine finished surfaces shall be protected against corrosion.

(d) Field painting is a phase of maintenance, but it is important that unless otherwise properly protected, all steel work shall after erection be protected by a field coat of good paint applied by a competent painter.

22. **Erection.**—(a) The frame of all steel skeleton buildings shall be carried up true and plumb, and temporary bracing shall be introduced wherever necessary to take care of all loads to which the structure may be subjected, including erection equipment, and the operation of same. Such bracing shall be left in place as long as may be required for safety.

(b) As erection progresses the work shall be securely bolted up to take care of all dead load, wind and erection stresses.

(c) Wherever piles of material, erection equipment, or other loads are carried during erection, proper provision shall be made to take care of stresses resulting from the same.

(d) No riveting shall be done until the structure has been properly aligned.

(e) Rivets driven in the field shall be heated and driven with the same care as those driven in the shop.

23. **Inspection.**—(a) Material and workmanship at all times shall be subject to the inspection of experienced engineers representing the purchaser.

(b) Material or workmanship not conforming to the provisions of this Specification shall be rejected at any time defects are found during the progress of the work.

(c) The Contractor furnishing such material or doing such work shall promptly replace the same.

(d) All inspection as far as possible shall be made at the place of manufacture, and the Contractor or Manufacturer shall co-operate with the Inspector, permitting access for inspection to all places where work is being done.

REFERENCES.—For the details of the design of tall buildings the following books may be consulted: Kidder's "Architects and Builders Pocketbook"; Freitag's "Fire Prevention and Fire Protection"; Freitag's "Architectural Engineering"; Ketchum's "The Design of Steel Mill Buildings."

For a full discussion of foundations for steel office buildings, see Jacoby and Davis, "Foundations of Bridges and Buildings," published by McGraw-Hill Book Co.

CHAPTER III.

STEEL HIGHWAY BRIDGES.

Definition.—A truss is a framework composed of individual members so fastened together that loads applied at the joints produce only direct tension or compression. The triangle is the only geometrical figure in which the form is changed only by changing the lengths of the sides. In its simplest form every truss is a triangle or a combination of triangles. The members of the truss are either fastened together with pins, pin-connected, or with plates and rivets, riveted.

Types of Truss Bridges.—The bridge in Fig. 1 consists of two vertical trusses which carry the floor and the load; of two horizontal trusses in the planes of the top and bottom chords, respectively, which carry the horizontal wind load along the bridge, and of cross-bracing in the planes of the end-posts, called portals, and in the planes of the intermediate posts, called sway bracing.

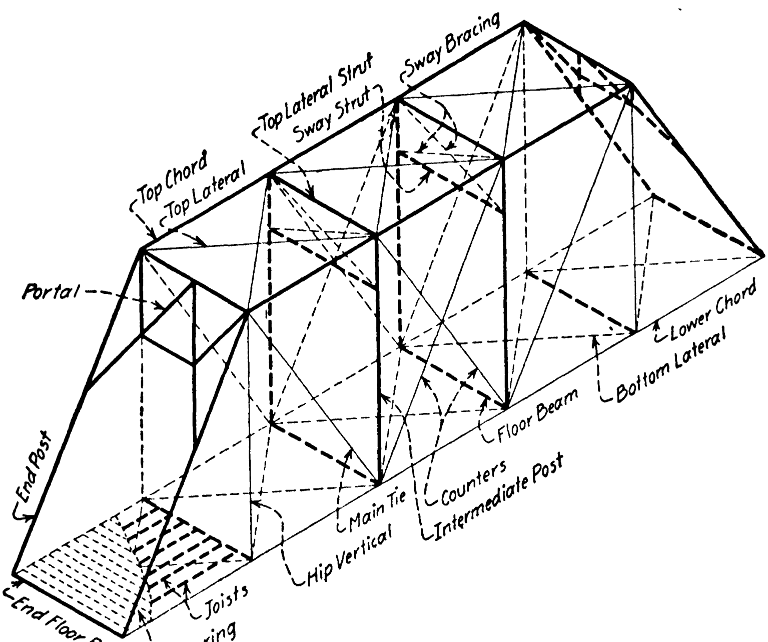


FIG. 1. DIAGRAMMATIC SKETCH OF A THROUGH PRATT TRUSS HIGHWAY BRIDGE.

The floor is carried on joists or stringers placed parallel to the length of the bridge, and which are supported in turn by the floorbeams. The names of the different parts of the bridge are shown in Fig. 1. The main ties, hip verticals, counters and intermediate posts are together called "webs." The bridge shown in Fig. 1, is a through pin-connected highway bridge of the Pratt type, the traffic passing through the bridge. In a deck bridge the roadway floor is carried on top of the main trusses. The bridge shown has square abutments; if the abutments are not at right

angles to the center line the bridge is called a "skew" bridge. Short span highway and railway bridges have low trusses and no top lateral system nor portals, as in Fig. 2. In a railway bridge the loads are carried to the panel points by stringers resting on or riveted to the floorbeams.

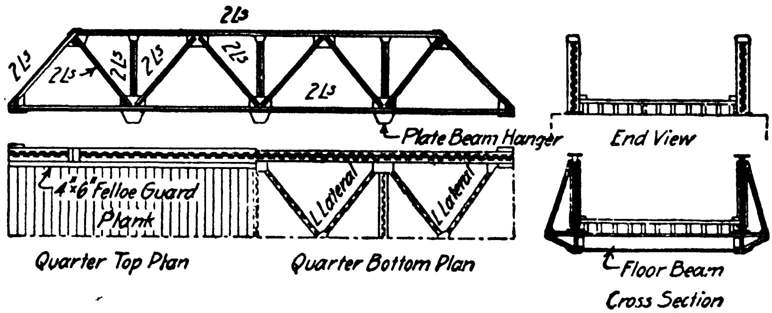


FIG. 2. PLAN OF A LOW OR "PONY" TRUSS HIGHWAY BRIDGE.

The simplest type of bridge is the beam bridge, (a) Fig. 3. Beam bridges commonly consist of I beams which span the opening, and are placed near enough together to carry the floor of the

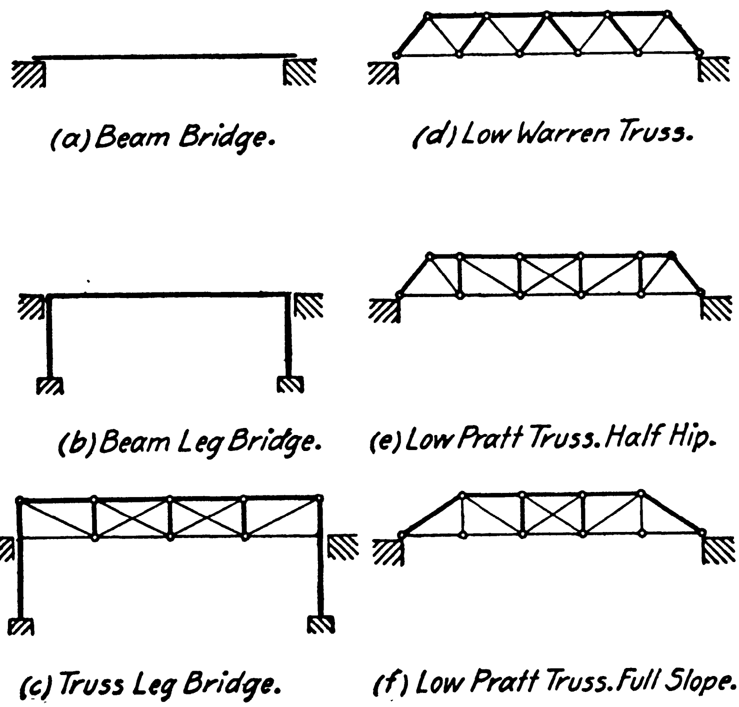


FIG. 3. TYPES OF SHORT SPAN HIGHWAY BRIDGES.

bridge. Where foundations are relatively expensive the beams may be carried on posts as in (b), Fig. 3. A truss leg-bridge is shown in (c), Fig. 3. Types (b) and (c) unless constructed with great care make inferior structures and are not to be recommended. A Warren truss is a combi-

nation of isosceles triangles as shown in (d), Fig. 3 and in (c) and (d), Fig. 4. The Pratt truss has its vertical web members in compression while its diagonal web members are in tension, as shown in (b), Fig. 4. The Warren truss is commonly built with riveted joints while the Pratt truss is usually built with pin-connected joints. The Warren low truss with riveted joints as shown in (d) is generally preferred in place of the low Pratt truss in either (e) or (f), Fig. 3. The Howe truss has its vertical web members in tension, and its inclined web members in compression as shown in (a), Fig. 4. The upper and lower chords and the inclined members of a Howe truss are commonly made of timber, while the vertical tension members are iron or steel rods or bars.

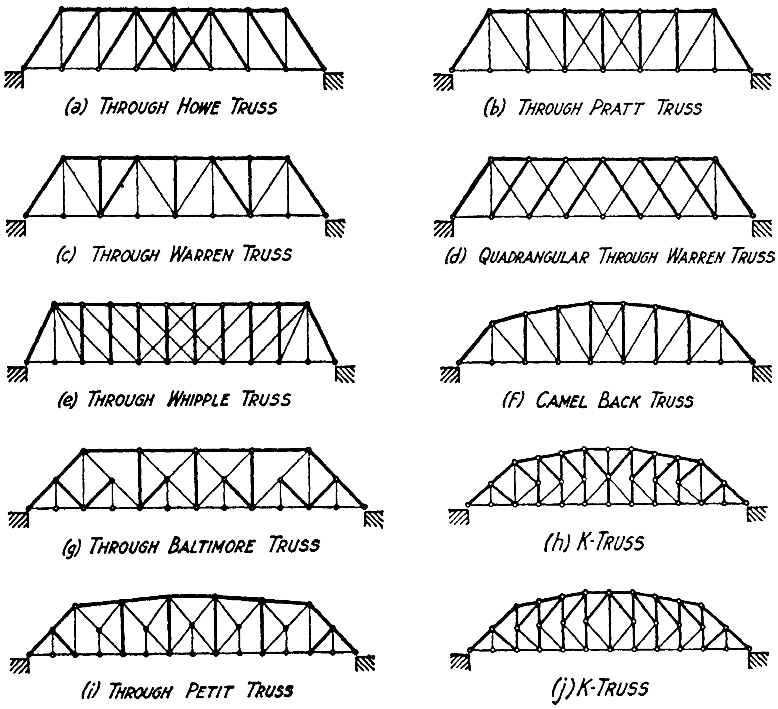


FIG. 4. TYPES OF HIGH TRUSS STEEL BRIDGES.

The Whipple truss, (e) Fig. 4, is a double intersection Pratt truss. This truss was designed to give short panels in long spans which have a considerable depth. The stresses in the Whipple truss are indeterminate for moving loads, and its use has been practically abandoned, the Baltimore truss, (g) Fig. 4 being used in its place. The quadrangular Warren truss with riveted joints is used by the American Bridge Company as a standard truss for through highway bridges, with spans of from 80 to 170 ft. Like the Whipple truss its stresses are indeterminate for moving loads.

For spans of from, say, 170 to 240 ft. it is quite common to use pin-connected trusses of the Pratt type having inclined chords as in (f), Fig. 4. The K-bracing in (h) or (j) is more economical of material and gives smaller secondary stresses than the subdivided bracing in (g) and (i), and is rapidly replacing both forms of bracing shown.

The Baltimore truss, (g) Fig. 4, is a Pratt truss with parallel chords in which the main panels have been subdivided by an auxiliary framework. The auxiliary framework may have struts as in (g), or ties as in (i), Fig. 4. The Baltimore truss with inclined upper chords, (i) Fig. 4, is

called a Petit truss. Baltimore and Petit trusses are statically determinate for all conditions of loading; are economical in construction and satisfactory in service, and have almost entirely replaced the Whipple truss for long span bridges.

The types of simple bridge trusses described above are those that are in the most common use, although quite a number of other types of trusses have been used and abandoned.

Beams and Plate Girders.—For spans of, say, 30 ft. and under rolled beams are often used to carry the roadway, while for spans from about 30 to 100 ft. plate girders are used for city bridges. When the roadway is carried on top of the girders, the bridge is called a deck plate girder bridge, and when the roadway passes between the girders, the bridge is called a through plate girder bridge as in Fig. 19.

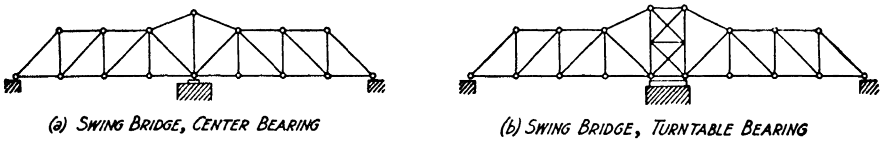


FIG. 5. SWING BRIDGES.

Swing Bridges.—Swing bridges may be made of plate girders or trusses, and may turn on a center pivot as in (a), or on a turntable supported on a drum as in (b), Fig. 5. The center pivot swing bridge has two spans continuous over the pivot support, while the turntable swing bridge has three spans ordinarily continuous over the middle supports.

Steel Arches.—Steel arch bridges are made (1) with three hinges, (2) with two hinges, and (3) without hinges, and may have solid webs, or spandrel or open webs.

Cantilever Bridges.—A cantilever bridge consists of two anchor spans, which support a suspended or channel span. The shore ends of the anchor spans are anchored to the shore piers and are supported on the river piers.

Suspension Bridges.—In a suspension bridge the roadway is supported by hangers attached to the main cables. Stiffening trusses are placed above the plane of the roadway to assist in distributing the live loads and for the purpose of increasing the rigidity of the structure.

Simple truss bridges, beam and plate girder bridges, only, will be considered in this book.

TYPES OF STRUCTURE.—The types of structure for steel highway bridges as recommended by the author are given in section 3, "General Specifications for Steel Highway Bridges," printed in the last part of this chapter.

The following data will show present standard practice.

Illinois Highway Commission.—The types of highway bridge recommended by the commission are as follows:

Concrete Bridges.—For culverts requiring a waterway of 12 square feet or less, plain or reinforced concrete arch culverts or square culverts, reinforced concrete pipes or double strength cast-iron pipe.

For culverts having an area of more than 12 square feet, and for bridges having a span up to 30 ft., reinforced concrete slabs, plain or reinforced concrete arches.

For spans of 30 ft. to 65 ft., reinforced concrete through or deck girders, plain or reinforced concrete arches.

For spans greater than 65 ft., plain or reinforced concrete arches.

Steel Bridges.—For spans of 12 ft. to 45 ft., steel I-beams; for spans of 30 ft. to 100 ft., plate girders or riveted pony trusses; for spans of 90 ft. to 160 ft., riveted trusses with parallel chords; for spans of 160 ft. and more, riveted or pin-connected trusses with parallel or inclined upper chords.

Iowa Highway Commission.—The types of highway bridges recommended by the commission are as follows:

Concrete Bridges.—Box culverts for spans up to 16 ft.; slab bridges for spans from 14 ft. to 25 ft.; arch culverts and bridges for spans of 6 ft. and over; girder bridges for spans of from 24 ft. to 40 ft.

Steel Bridges.—Steel I-beams up to 32 ft. span; plate girders, 20 ft. to 80 ft. span; low truss 30 ft. to 100 ft. span; high truss 100 ft. span and over, riveted up to 140 ft. span.

Massachusetts Public Service Commission.—The types of highway bridge recommended by the commission are as follows:

Steel Bridges.—For spans up to 20 ft., wooden stringers or rolled beams; for spans from 20 ft. to 40 ft., rolled beams or plate girders; for spans from 40 ft. to 70 ft., plate girders; for spans from 70 ft. to 100 ft., plate girders or riveted trusses; for spans from 100 ft. to 125 ft., riveted trusses; for spans from 125 ft. up, riveted or pin trusses.

Wisconsin Highway Commission.—The types of highway bridge recommended by the commission are as follows:

Concrete Bridges.—Spans of 1½ ft. to 10 ft., slab culverts and bridges; spans 10 ft. to 18 ft., slab bridges; spans 10 ft. to 40 ft., through girders.

Steel Bridges.—Spans 10 ft. to 38 ft., rolled beams; spans 35 ft. to 80 ft., Warren riveted low trusses or plate girders; spans 80 ft. to 135 ft., Pratt riveted high trusses; spans over 135 ft., riveted high trusses with curved chords.

WIDTH OF ROADWAY.—The following data will show standard practice.

Illinois Highway Commission.—The widths of roadways are specified for State Aid Routes, Principally Traveled Roads, and Secondary Roads.

On Designated State Aid Routes.—Bridges up to and including 10 ft. span, 20 to 30 ft. roadway; bridges over 10 ft. up to and including 60 ft. span, 18 to 24 ft. roadway; bridges over 60 ft. span, 16 to 20 ft. roadway.

On Principally Traveled Roads.—Bridges and culverts 10 ft. or less in span, 20 to 30 ft. roadway; bridges over 10 ft. and up to and including 60 ft. span, 16 to 20 ft. roadway; bridges over 60 ft. span, 16 to 18 ft. roadway.

On Secondary Roads.—Bridges and culverts 10 ft. or less in span, 18 to 24 ft. roadway; bridges over 10 ft. span, 16 ft. roadway.

Culverts Under Fills.—The length of the barrel of the culvert shall have a length that will permit of side slopes of 1½ horizontal to 1 vertical, and a top width of 20 to 30 ft. on State Aid Routes, 20 to 30 ft. on Principally Traveled Roads, and 18 to 24 ft. on Secondary Roads.

Iowa Highway Commission.—The widths of roadway for highway bridges as recommenedd by the commission are as follows:

Concrete Bridges.—For box or arch culverts with spans of 2 ft. to 16 ft., 24 ft. roadway for county roads, and 20 ft. for township roads; for slab bridges with spans over 16 ft. span, 20 ft. roadway for county roads, and 18 ft. for township roads; for girder bridges over 16 ft. span, 20 ft. roadway; for arches over 16 ft. span, 24 ft. roadway for county roads, and 20 ft. for township roads. The slopes on fills shall be 1½ horizontal to 1 vertical.

Steel Bridges.—A roadway of 20 ft. on county roads, for all spans, and 18 ft. on township roads for all spans. The minimum legal width of roadway is 16 ft.

Association of State Highway Departments.—The following minimum widths of concrete bridges are recommended.

For First Class Roads.—Culverts under 12 ft. span, 24 ft. roadway; slab bridges over 12 ft. span, 20 ft. roadway; all other spans 20 ft. roadway.

For Second Class Roads.—Culverts under 12 ft. span, 20 ft. roadway; slab bridges over 12 ft. span, 18 ft. roadway; all other spans, 18 ft. roadway.

For Third Class Roads.—Culverts under 12 ft. span, 20 ft. roadway; slab bridges over 12 ft. span, 18 ft. roadway; longer bridges, 16 ft. roadway.

The above widths of concrete bridges have been adopted by the Wisconsin Highway Commission.

LOADS.—The loads carried by a bridge consist of (1) fixed or dead loads, (2) the moving or live load, and (3) miscellaneous loads.

The dead load consists of the weight of the structure and is always carried by the bridge; the live load consists of the moving load which the bridge is built to carry, while the miscellaneous loads include wind loads, snow loads, etc. Data on dead loads are given in the "Specifications for Steel Highway Bridges" in the last part of this chapter.

WEIGHTS OF BRIDGES.—The weight of a bridge is composed of (1) the weight of the steel in the steel framework, consisting of the vertical trusses, the upper and lower lateral systems, the floorbeams, the portals and sway bracing; (2) the weight of the joists and the fence; and (3) the weight of the floor covering.

WEIGHTS OF STEEL HIGHWAY BRIDGES.—The following data may be used in calculating the dead loads in the design of highway bridges or as a basis for preliminary estimates.

AMERICAN BRIDGE COMPANY.—Standard Steel Highway Bridges with Timber Floor. Timber floor, 3-in. plank on roadway and 2-in. plank on footwalks. Live loads for floor and its supports, 100 lb. per sq. ft. of floor surface, or 6 tons on two axles 10 ft. centers and 5 ft. gage, or a 15-ton road roller. For trusses 100 lb. per sq. ft. of roadway up to a span of 75 ft., 75 lb. per sq. ft. of roadway for spans of 168 ft. and over, and proportional for intermediate spans. No allowance is made for impact. Designed for allowable stresses given in specifications in the latter part of this chapter. Let W = weight of the structural steel per lineal foot of span; L = length of span in feet, b = width of roadway in feet (without sidewalks).

1. **Steel Through Plate Girders.**—Through plate girder spans 36 ft. to 70 ft., roadway 20 ft. wide, without sidewalks, but including stringers. The weight of structural steel per lineal foot of span is

$$W = 300 + 3.8L. \tag{1}$$

For sidewalks with steel joists add about 12 lb. per sq. ft. of sidewalks.

2. **Steel Low Riveted Truss Spans, with Timber Floor.**—For low truss spans 36 ft. to 102 ft., with timber floors, the weight of structural steel per lineal foot of span, not including the weight of the stringers and the railing, is given approximately by the formula for a 16-ft. roadway

$$W = 100 + 2.0L. \tag{2}$$

and for a 20-ft. roadway

$$W = 150 + 1.7L. \tag{3}$$

3. **Steel Low Riveted Truss Spans, with Reinforced Concrete Floors.**—For low truss spans 36 ft. to 102 ft., with reinforced concrete floors, 5 in. thick with 6 in. of gravel at center and 3 in. of gravel at curb, the weight of structural steel per lineal foot of span, not including the weight of the stringers and the railing, is given approximately by the formula for a 16-ft. roadway

$$W = 150 + 3.5L. \tag{4}$$

and for a 20-ft. roadway

$$W = 185 + 3.5L. \tag{5}$$

4. **Steel High Truss Spans, with Timber Floor.**—For high truss spans 104 to 204 ft., with timber floors the weight of structural steel per lineal foot of span, not including the weight of the stringers and the railing, is given approximately by the formula for a 16-ft. roadway

$$W = 250 + 1.5L. \tag{6}$$

and for a 20-ft. roadway

$$W = 285 + 1.2L. \tag{7}$$

IOWA HIGHWAY COMMISSION.—Steel Highway Bridges with Reinforced Concrete Floor.—Reinforced concrete floor slabs 6 in. thick for all spans in which stringers are used. Slabs for stringerless floors 7½ in. thick for 8-ft. span, 8 in. thick for 9-ft. span, and 8½ in. thick for 10-ft. span. Live loads for the floor and its supports a uniform live load of 100 lb. per sq. ft., and a 15-ton traction engine with two-thirds of the load on the rear axle; axles spaced 11 ft. centers, and rear wheels spaced 6 ft. centers. Rear wheels 22 in. wide. The trusses are to be designed for the uniform loads given in Table I. No allowance is made for impact.

Let W = weight of structural steel in lb. per lineal foot of span; L = length of span in feet; b = width of span in feet (without sidewalks).

1. **Steel Beam Spans.**—The weight of steel beam spans from 16 ft. to 32 ft. and with 16-ft., 18-ft., and 20-ft. roadway are given in Table IX.

2. Steel Low Truss Spans, with Stringers.—For low truss highway bridges with spans of 35 ft. to 85 ft., not including the weight of the fence or the steel stringers, the weight of structural steel per lineal foot of span for a 16-ft. roadway is

$$W = 235 + 2.35L. \tag{8}$$

and for an 18-ft. roadway is

$$W = 240 + 2.40L. \tag{9}$$

3. Steel Low Truss Spans, without Stringers.—For low truss highway bridges with spans of 35 ft. to 100 ft., not including the weight of the fence or steel floorbeams, the weight of the structural steel per lineal foot of span for a 16-ft. roadway is

$$W = 200 + 4L. \tag{10}$$

and for an 18-ft. roadway is

$$W = 225 + 4.25L. \tag{11}$$

4. Steel High Truss Spans, with Stringers.—For high through truss highway bridges with spans of from 90 ft. to 150 ft., not including the weight of fence or the steel stringers, the weight of structural steel per lineal foot of span for a 16-ft. roadway is

$$W = 245 + 2.45L. \tag{12}$$

and for an 18-ft. roadway is

$$W = 270 + 2.7L. \tag{13}$$

WISCONSIN HIGHWAY COMMISSION. Steel highway bridges with reinforced concrete floor.—Reinforced concrete floor slabs 6 in. thick for all spans. Live loads for the floor and its supports a 15-ton road roller with two-thirds of the load on the rear axle, axles 10 ft. centers, rear rolls 4 ft. 10 in. centers, rear rolls 20 in. wide. The trusses designed for the loads given in Table I. No allowance is made for impact. Let W = weight of structural steel in lb. per lineal foot of span, L = length of span in feet; b = width of roadway in feet (without sidewalks).

1. Steel Beam Spans.—Weight of steel beam spans from 10 ft. to 38 ft. and for 16-ft., 18-ft. and 20-ft. roadway are given in Table X.

2. Steel Through Plate Girders.—The weight of the structural steel in through plate girder highway bridges from 35 ft. span to 80 ft. span including floorbeams spaced 3 to 2½ ft. apart, is given approximately by the following formula. For a 16-ft. roadway

$$W = 300 + 3L. \tag{14}$$

For an 18-ft. roadway

$$W = 300 + 3.25L. \tag{15}$$

and for a 20-ft. roadway

$$W = 320 + 4L. \tag{16}$$

3. Steel Low Truss Spans, with Stringers.—The weight of the structural steel in low truss steel highway bridges with spans of 35 ft. to 85 ft. span, not including the weight of the fence or the steel stringers, is given approximately by the formula. For a 16-ft. roadway

$$W = 80 + 3.5L. \tag{17}$$

and for an 18-ft. roadway

$$W = 80 + 4L. \tag{18}$$

4. Steel High Truss Spans, with Stringers.—For high through truss steel highway bridges with spans of from 90 ft. to 150 ft., not including the weight of the fence or the steel joists, the weight of structural steel per lineal foot of span is given approximately by the formula. For a 16-ft. roadway

$$W = 180 + 2L. \tag{19}$$

and for an 18-ft. roadway

$$W = 240 + 2L. \tag{20}$$

ILLINOIS HIGHWAY COMMISSION. Steel highway bridges with reinforced concrete floor.—Reinforced concrete floor slabs 4 in. thick with a wearing surface assumed to weigh not less than 50 lb. per sq. ft. Live load for floor and its supports a 15-ton traction engine, supported on two axles spaced 10 ft. apart, with two thirds of the load on the rear axle; or a uniform live load of 125 lb. per sq. ft. The trusses designed for the loads given in Table I. No allowance is made for impact.

Let W = weight of steel in lb. per lineal foot of span, L = span of bridge in feet, b = width of roadway in feet (without sidewalks).

1. **Steel Low Truss Spans, with Stringers.**—The weight of the structural steel in low truss steel highway bridges with spans of 50 ft. to 85 ft., not including weight of the fence or the steel stringers, is given approximately by the formula. For a 16-ft. roadway, b = 16 ft

$$W = 235 + 2.35L. \tag{21}$$

and for an 18-ft. roadway, b = 18 ft.

$$W = 240 + 2.4L. \tag{22}$$

2. **Steel High Truss Spans, with Stringers.**—The weight of structural steel in high truss steel highway bridges with spans of 90 ft. to 160 ft., not including the weight of fence or the steel stringers, is given approximately by the formula. For a 16-ft. span, b = 16 ft.

$$W = 140 + 4L. \tag{23}$$

and for an 18-ft. span, b = 18 ft.

$$W = 180 + 4.5L. \tag{24}$$

The weights given by formulas (21) to (24) are for bridges with concrete floors weighing 100 lb. per sq. ft. Calculations by Mr. Clifford Older, Bridge Engineer, Illinois Highway Commission, show that a variation of the weight of the floor of 10 lb. per sq. ft. makes a similar variation in the weight of the structural steel, including the joists, of 4.35 per cent for a 50-ft. span, of 3.75 per cent for a 160-ft. span, and proportional for intermediate spans. For the structural steel, not including the joists, an average value of 4 per cent may be used for each decrease of 10 lb. per sq. ft. of floor surface.

BOSTON BRIDGE WORKS STANDARDS.*—The weights of steel highway bridges designed by the Boston Bridge Works are as follows:

Through truss highway bridges without sidewalks designed for a live load of 80 lb. per sq. ft. for the trusses, 100 lb. per sq. ft. and a 6-ton wagon for the floor The weight, w, of steel in lb. per sq. ft. of area covered by the floor, not including joist or fence, for a span of L ft., is

$$w = 5 + L/9.5 \tag{25}$$

The weight of through truss highway bridges with two sidewalks is

$$w = 2.8 + L/11.3 \tag{26}$$

The sidewalks were 5 or 6 ft. wide, and the clear roadways were 16 to 20 ft. The total area covered by the roadway and sidewalk floors is to be used in calculating the weight of steel.

Weights of Steel Highway Plate Girder Bridges.—The weights of highway plate girder bridges as designed by the Boston Bridge Works for the live loads shown are as follows.

Deck plate girder highway bridges without sidewalks designed for a live load of 100 lb. per sq. ft. for girders, 100 lb. per sq. ft. and a 6-ton wagon for the floor. The weight, w, of steel in lb. per sq. ft. of area covered by the floor, not including joist or fence, for a span of L ft., is

$$w = 2.5 + L/3.4 \tag{27}$$

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The weight of deck plate girder highway bridges with sidewalks is

$$w = 2.5 + L/4.4 \tag{28}$$

The weight of through plate girder highway bridges without sidewalks is

$$w = 3 + L/4.25 \tag{29}$$

The weight of through plate girder highway bridges with sidewalks is

$$w = 3.3 + L/5.6 \tag{30}$$

Weight of Electric Railway Bridges.—The Boston Bridge Works gives the following formula for the weight of electric railway bridges, where W = total weight of steel in lb. per lineal foot of bridge and L is the span of the bridge in feet.

Beam bridges

$$W = 50 + 5L \tag{31}$$

Light truss bridges

$$W = 200 + 0.8L \tag{32}$$

Heavy truss bridges

$$W = 250 + 1.5L \tag{33}$$

The beam bridges were designed for 30-ton cars; the light truss bridges were designed for 15-ton cars or 1,500 lb. per lineal foot of bridge, and the heavy truss bridges were designed for 30-ton cars, or 2,000 lb. per lineal foot of bridge.

LIVE LOADS.—The live loads for highway bridges are usually assumed to consist of a uniform live load for the trusses and a uniform live load or a concentrated moving load for the floor and its supports. A few highway bridge specifications require that trusses be designed for a concentrated moving load as well as for a uniform live load, and also that the floor and its supports be designed for a concentrated moving load and that the portion of the floor of the bridge not covered by the concentrated load be covered with a uniform live load. In calculating the stresses in the truss members the uniform live load is commonly assumed as applied in full joint loads at joints on the loaded chord. Moving loads and loads suddenly applied produce stresses that are greater than the static stresses due to stationary loads or to loads gradually applied. This increase in stress due to moving loads or due to loads suddenly applied is called impact stress.

IMPACT.—The effect of impact or increase in live load stresses over the stresses due to the same loads gradually applied, is very much less for highway bridges than for railway bridges. Experiments made by Professor F. O. Dufour and recorded in Journal of Western Society of Engineers, June, 1913, show that the effect of impact on steel truss highway bridges with concrete floors is very small. The effect of impact on steel truss bridges with plank floors is considerably larger than for bridges with concrete floors. The maximum impact percentages do not occur with maximum static stresses. Experiments made at the University of Colorado under the author's direction show that the effect of impact on highway bridges is very much less than for railway bridges.

Tests now (1924) being carried on at Iowa State College with the cooperation of the Iowa State Highway Commission and the U. S. Bureau of Public Roads show that moving trucks, motor cars and tractors produce a much greater impact than was generally believed. While the investigation has not been completed, the tests thus far would appear to indicate (1) that the impact effect on the floorbeam hangers is very much greater than on the stringers, (2) that the impact on the truss members is less than on the stringers, (3) that impact on a highway bridge with concrete floor is probably greater than on a bridge with a timber floor. Based on these tests the Iowa State Highway Commission has adopted the following specification for impact in its 1923 Specifications for Highway Bridges. **"For I-beam spans and floor systems of trusses and girders the impact shall be taken as 33⅓ per cent; for floorbeam hangers the impact shall be taken as 66⅔ per cent; for trusses and through girders the impact shall be given by the formula $I = 75/(L + 200)$ where L = loaded length of span in feet."**

The "Specifications for Steel Highway Bridges" adopted by the American Association of State Highway Officials in 1923, require that impact shall be calculated by the formula

$$I = \frac{L + 250}{10L + 500}.$$

The bridge committee of the American Society of Civil Engineers in the tentative "Specifications for Steel Highway Bridges" presented to the Society June 25, 1923, adopted the following specification for impact on highway bridges. "For floorbeams and stringers I = 30 per cent, on floorbeam hangers I = 60 per cent. For girders and trusses, $I = 100/(L + 300)$ where L = loaded length of span in feet."

Ketchum's Specifications for Impact.—The author has adopted the following impact factors for concrete bridges and steel bridges.

(a) For concrete arches with spandrel filling or culverts with a minimum filling of one foot, no allowance for impact.

(b) For concrete slab and girder bridges and trestles, and arches without spandrel filling, 30 per cent for impact.

(c) For steel bridges the following allowance for impact. For the floor and its supports including floor slabs, floor joist, floorbeams and hangers, 30 per cent.

For all truss members other than the floor and its supports, the impact increment shall be $I = 100/(L + 300)$, where L = length of span for simple highway spans (for trestle bents, towers, movable bridges, arch and cantilever bridges, and for bridges carrying electric trains, L shall be taken as the loaded length of the bridge in feet producing maximum stress in the member).

CONCENTRATED LIVE LOADS.—Traction engines weighing 20 tons are quite common in the west and northwest. The heaviest motor truck in common use has a capacity of $7\frac{1}{2}$ tons and a total weight of 13 tons, with nearly 10 tons on the rear axle. With an overload of 50 per cent, which is not unusual, this truck would carry 14 tons on the rear axle. The maximum road roller weighs 20 tons.

The Association of State Highway Officials in "Specifications for Steel Highway Bridges," adopted in 1923, specifies the following live loads. Class A bridges (bridges on primary roads) with roadway 18 ft. or more are to be designed for two 15-ton trucks (2–15). Class B bridges (light traffic bridges) are to be designed for one 15-ton truck (1–15). Class C bridges (heavy traffic bridges) for roadway less than 18 ft., one 20-ton truck (1–20); for bridges over 18 ft., two 20-ton trucks (2–20). The standard truck is to have axles 14 ft. centers, wheels 6 ft. centers, 80 per cent of total load on rear axle, each rear tire to have one inch in width for each ton weight of truck.

The 1923 specifications of the Iowa State Highway Commission has adopted the above loadings, and has added a fourth. For unimportant bridges the live load on bridges with a roadway less than 18 ft. shall be one 10-ton truck (1–10), and for bridges with roadway more than 18 ft., one 15-ton truck (1–15).

The tentative 1923 "Specifications for Steel Highway Bridges" of the American Society of Civil Engineers specify the same live loads as American Association of State Highway Officials.

For additional data see article entitled "Concentrated Live Loads for Highway Bridges," by Milo S. Ketchum, printed in University of Colorado Journal of Engineering, October, 1916.

Ketchum's Specifications for Concentrated Moving Loads.—The author has adopted the following specifications for moving concentrated loads.

(a) That highway bridges on main roads or near towns or cities shall be designed to carry a 20-ton motor truck with axles spaced 12 ft. and wheels with a 6-ft. gage, with 14 tons on rear axle and 6 tons on front axle. The truck to occupy a space 10 ft. wide and 32 ft. long. The rear wheels to have a width in inches equal to the total load in tons (20 in. for a 20-ton truck).

(b) That bridges not on main roads shall be designed for a 15-ton motor truck with axles spaced 10 ft. and wheels with a 6-ft. gage, and occupying a space 10 ft. wide and 30 ft. long, with 10 tons on rear axle and 5 tons on front axle, and with rear wheels 15 in. wide.

(c) To provide for impact and vibration and unevenness of road surface thirty (30) per cent is to be added to the maximum live load stresses. Only one motor truck is to be assumed to be on a bridge at one time.

Motor trucks have narrower tires and are driven at greater speeds than traction engines, and therefore not only produce greater static stresses in the floor, but should have a greater impact allowance. In view of the above, it would not appear to be necessary to consider any road rollers or traction engines now in use in addition to the above motor-truck loadings.

DISTRIBUTION OF CONCENTRATED LOADS.—In designing floor slabs, floor stringers and floorbeams it is necessary to know the distribution of the concentrated loads.

Concrete Floor Slabs.—Tests of the distribution of concentrated loads on concrete floor slabs have been made by the Ohio Highway Commission, the results of which are given in Bulletin No. 28, published by the Commission; by Mr. W. A. Slater at the University of Illinois and described in Proceedings of American Society for Testing Materials, Vol. XIII, 1913, and by A. T. Goldbeck and E. B. Smith, described in Journal of Agricultural Research, Vol. VI, No. 6, Department of Agriculture, Washington, D. C., May 8, 1916.

Ohio Tests.—The following conclusions drawn from the Ohio tests are of interest:

"The percentage of reinforcement has little or no effect upon the distribution to the joists, so long as safe loads on the slabs are not exceeded.

"The outside joists should be designed for the same total live load as the intermediate joists.

"The axle load of a truck may be considered as distributed over 12 ft. in width of roadway.

"The safe value for 'effective width' of a slab, where the total width of slab is greater than $1.33\,L + 4$ ft. is given by the formula, $e = 0.6L + 1.7$ ft., where e = effective width (width over which a single concentrated load may be considered as uniformly distributed on a line down the middle of the slab parallel to the supports) and L = span in feet."

Slater Tests.—It was recommended that where the total width of slab is greater than twice the span, the effective width be taken as $e = 4x/3 + d$, where x is the distance from the concentrated load to the nearest support, and d is the width at right angles to the support over which the load is applied. While the depth of slab and the amount of longitudinal reinforcement had little effect on the distribution, it was recommended that the latter be limited to 1 per cent.

Goldbeck and Smith Tests.—Tests were made on three slabs, each slab being 32 ft. wide, 16 ft. span, and with effective depths of 10.5 in., 8.5 in. and 6 in., respectively. All slabs were made of 1-2-4 Portland cement concrete, and were reinforced with 0.75 per cent of mild steel.

The following conclusions were drawn from these tests:

(1) The effective width decreases as the effective depth increases; the effective width for safe loads being 75.7 per cent; 81.1 per cent, and 109.3 per cent of the span, for the slabs having effective depths of 10.5 in., 8.5 in. and 6 in., respectively.

(2) For slabs in which the ratio of the width of the slab is not less than twice the span length, the effective width may be taken as

$$e = 0.7L \tag{34}$$

where e is the effective width and L is the span length.

(Additional tests by Goldbeck, Proceedings American Concrete Institute, 1917, show that formula (34) may be used when the width of the slab is not less than the span.)

Watson's "General Specifications for Concrete Bridges," third edition, 1916, specifies that concentrated loads on reinforced concrete slabs may be assumed as distributed over a distance of 4 ft. at right angles to the supports, and a distance parallel to the supports equal to 2 ft. plus three-tenths of the span of the slab.

The State Highway Department of Ohio uses the following distribution of concentrated loads on floor slabs.

For spans less than 6 ft. the percentage, p, of the wheel load carried by one foot in width of slab for a span in feet, l, is given by the formula

$$p = 42 - 4l \tag{35}$$

while for spans greater than 6 ft. the percentage, p', of the wheel load carried by one foot in width of slab for a span in feet, l, is given by the formula

$$p' = 20 - 0.4l \tag{36}$$

For a span of $5\frac{1}{2}$ ft., from formula (35), p = 20 per cent, and the concentrated load is assumed as carried by a slab 5 ft. wide, applied on a line parallel to the supports.

For a span of 10 ft., from formula (36), p' = 16 per cent, and the concentrated load is assumed as carried by a slab 6.67 ft. wide, applied on a line parallel to the supports.

Floor Stringers and Floorbeams.—The 1923 "Specifications for Steel Highway Bridges" adopted by the American Association of State Highway Officials, the 1923 "Specifications for Steel Highway Bridges" of the American Society of Civil Engineers, and the 1923 "Specifications for Steel Highway Bridges" of the Iowa State Highway Commission contain the following specifications for the distribution of loads to floor stringers and floorbeams.

Bending Moment in Stringers.—In determining bending moments in stringers, each wheel load shall be assumed to be concentrated at a point.

When the floor system is designed for one truck, each interior stringer shall be proportioned to support that part of one rear-wheel load, or those parts of one front-wheel load and one rear-wheel load, represented by a fraction the numerator of which is the stringer spacing, in feet, and the denominator of which is 4 ft. for plank floors; 5 ft. for 4-in. and 6-in. strip floors and wood blocks on a 4-in. plank sub-floor; 6 ft. for reinforced concrete floors.

When the floor system is designed for two trucks, the corresponding lengths shall be: 3 ft. 6 in. for plank floors; 4 ft. for 4-in. and 6-in. strip floors and wood blocks on a 4-in. plank sub-floor; 4 ft. 6 in. for reinforced concrete floors. When the stringer spacing is greater than this distance, the stringer loads shall be determined by assuming the flooring between stringers to act as simple beams.

The live load supported by the outside stringers shall in no case be less than would be required for interior stringers.

Bending Moment in Floorbeams.—In determining bending moments in floorbeams, each wheel load shall be assumed as concentrated at a point. When stringers are omitted and the floor is supported directly on the floorbeams, the latter shall be proportioned to carry that fraction of one axle load, when the floor system is designed for two trucks, the numerator of which is the floorbeam spacing, in feet, and the denominator of which is: 4 ft. for plank floors; 5 ft. for 4-in. and 6-in. strip floors and wood blocks on a 4-in. plank sub-floor; 6 ft. for reinforced concrete floors. When the spacing of floorbeams exceeds the denominator given, but is less than the axle spacing (14 ft.), each beam shall be proportioned to carry the full axle load or loads.

Ketchum's Specifications for Distribution of Concentrated Loads.—From a study of the various tests and specifications, the author has adopted the following rules for calculating the stresses in slabs, stringers and floorbeams:

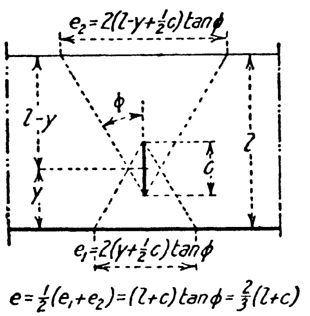


FIG. 6.

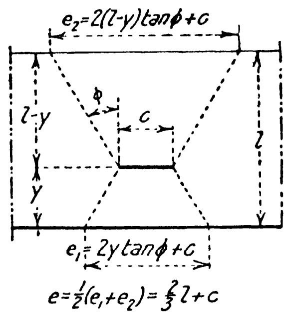


FIG. 7.

(*a*) The distribution of concentrated wheel loads for bending moments in reinforced concrete slabs with longitudinal girders shall be calculated by the formula,

$$e = \tfrac{2}{3}(l + c) \tag{37}$$

with a maximum limit of 6 ft. for e, where e = effective width (distance that the load may be considered as uniformly distributed on a line down the middle of the slab parallel to the supports), l = span, and c = width of tire of wheel, all distances in feet. See Fig. 6.

(*b*) The distribution of concentrated wheel loads for bending moments in reinforced concrete slabs with transverse girders shall be calculated by the formula

$$e = 2l/3 + c \tag{38}$$

with a maximum limit of 6 ft. for e, where e = effective width, l = span, and c = width of tire of wheel as defined in paragraph (a). See Fig. 7.

(*c*) The distribution of concentrated wheel loads for bending moments in slabs of girder bridges in which the span of the bridge is not less than the width of bridge center to center of girders, shall be calculated for spans of 9 ft. or over by the formula

$$e = 2l/3 \tag{39}$$

with a maximum limit of e = 12 ft., where e = effective width, and l = span as defined in paragraph (a).

(*d*) The effective width for shear in beams carrying concentrated loads shall be taken the same as for bending moment as calculated by formula (37) or formula (38), with a minimum effective width of 3 ft. and a maximum effective width of 6 ft.

The total shear for an effective width of 3 ft. shall be considered as punching (pure) shear. The total shear for an effective width of 4.5 ft. and over shall be considered as beam shear (a measure of diagonal tension), for effective widths between 3 ft. and 4.5 ft. the total shear shall be divided proportionally between punching shear and beam shear. Beam shear shall be used in calculating bond stress and as a measure of diagonal tension.

(*e*) In the design of longitudinal joists or stringers with concrete floors, the fraction of the concentrated load carried by one stringer for spacings 6 ft. or less will be taken equal to the stringer spacing in feet divided by 6 ft.; with plank floors the fraction of the concentrated load carried by one stringer for spacings 4 ft. or less will be taken equal to the stringer spacing in feet divided by 4 ft., the maximum in each case being the full load. Outside stringers are to be designed for the same load as intermediate stringers.

(*f*) In the design of transverse stringers or floorbeams with concrete floors, the fraction of the concentrated load carried by one floorbeam for floorbeams spaced 6 ft. or less, will be taken equal to the floorbeam spacing divided by 6 ft. For floorbeams spaced 6 ft. or over the entire reactions are assumed as carried by one floorbeam. Axle loads are assumed as distributed on a line 12 ft. long.

UNIFORM LIVE LOADS FOR TRUSSES.—The uniform live loads for trusses of steel highway bridges as specified by the highway commissions of Illinois, Iowa and Wisconsin, the American Concrete Institute, 1916, and the uniform loads as specified by the author for classes D_1 and D_2 are given in Table I. The D_1 and D_2 loadings are to be taken as proportional for intermediate spans, and are to be increased for impact.

It will be seen that the D_1 loadings with impact added are practically the same as the Illinois loadings; while the D_2 loadings with impact added are practically the same as the Iowa and Wisconsin loadings.

TABLE I.

UNIFORM LIVE LOADS FOR HIGHWAY BRIDGES.

Illinois Highway Commission.		Iowa Highway Commission.		Wisconsin Highway Commission.		American Concrete Institute, 1916.				Ketchum's Specifications, 1918.			
						Class A.		Class B.		Class D_1.		Class D_2.	
Span, Ft.	Load, Lb./Sq. Ft.	Span, Ft.	Load, Lb./Sq. Ft.	Span, Ft.	Load, Lb./Sq. Ft.	Span, Ft.	Load, Lb./Sq. Ft.	Span, Ft.	Load, Lb./Sq. Ft.	Span, Ft.	Load, Lb./Sq. Ft.	Span, Ft.	Load, Lb./Sq. Ft.
Up to 50	125	Up to 50	100	Up to 40	125	Up to 80	125	Up to 80	100	30	125	30	100
50–100	100	50–100	90	50	120	80–100	110	80–100	90	50	106	50	90
100–150	100	100–150	80	75	106	100–125	100	100–125	80	80	85	80	75
150–200	85	150–200	70	100	93	125–150	90	125–150	75	100	80	100	71
Over 200	85	200–250	50	150	60	150–200	85	150–200	65	160	68	160	60
		Over 250	50	180 and over	50	Over 200	80	Over 200	60	200 and over	60	200 and over	50

Iowa State Highway Commission, Class D_1 and D_2 bridge loadings, to be increased for impact.

Highway Live Loads for Girders and Trusses.—The 1923 highway bridge specifications of the American Association of State Highway Officials, the Iowa State Highway Commission, and the 1923 tentative specifications of the American Society of Civil Engineers contain the following specification for floor loads for girders and trusses, and for floors, as given in Table Ia.

The uniform load used shall correspond to the length of that portion of the span which, when fully loaded, will produce maximum stress in the member under consideration.

When the loaded length is less than 50 ft., girders and truss members shall be designed for the floor live load. The trucks shall be placed so as to produce the most severe stresses. Two trucks shall be considered as headed in the same direction. Trucks in tandem need not be considered.

TABLE Ia. UNIFORM LIVE LOADS FOR GIRDERS AND TRUSSES

Loaded Length, ft.	Live Load in lb. per sq. ft. Proportionate Values for Intermediate Lengths		
	1–15 Ton Truck	1–20 Ton Truck 2–15 Ton Truck	2–20 Ton Truck
50	100	130	180
100	80	90	120
200 and more	60	70	90

Sidewalk Live Loads.—Side walk live loads shall be 80 lb. per sq. ft. for loaded lengths of 50 ft., or less, and 60 lb. per sq. ft. for 100 ft., or more. For intermediate lengths proportionate loads shall be used.

Floor Live Loads.—All parts of the floor system and all girders and truss members when the loaded length is less than 50 ft., shall be designed for the following loads: (1–15) one 15-ton truck, or 100 lb. per sq. ft. of roadway; (1–20) one 20-ton truck, or 130 lb. per sq. ft. of roadway; (2–15) two 15-ton trucks; (2–20) two 20-ton trucks.

In bridges involving three or more lines of traffic, the floorbeams and floorbeam hangers shall be designed for two trucks assumed to be located in the most unfavorable position, together with a uniform live load of 100 lb. per sq. ft. on the remaining lines of roadway not occupied by the trucks.

DESIGN OF HIGHWAY BRIDGE FLOORS. Types of Floors.—The choice of floor for a highway bridge depends upon the traffic, the cost, including first cost and cost of maintenance, and the climate. A highway bridge floor consists of a sub-floor which has the necessary strength to carry the loads and a wearing surface. Plank floors and reinforced concrete slabs without wearing surface have the sub-floor and wearing surface combined. A highway bridge floor should have a strength and a weight appropriate to the structure of the bridge, and should be well drained. The wearing surface should be waterproof, capable of resisting wear and should be as smooth as possible without being slippery. For proper drainage the wearing surface should have a longitudinal grade of not less than 1 in 50 or a transverse slope of not less than 1 in 12. Sub-floors for highway bridges are made (1) of reinforced concrete; (2) of buckle plates or other steel sections, and (3) of timber. The most common wearing surfaces for highway bridge floors are (*a*) concrete, (*b*) bituminous concrete, (*c*) asphalt, (*d*) creosoted timber blocks, (*e*) brick, (*f*) stone block, (*g*) macadam, (*h*) gravel or earth. The different types of sub-floors and wearing surfaces for highway bridges will be described in some detail.

Reinforced Concrete Floor Slabs.—Reinforced concrete floor slabs on steel highway bridges may be supported on joists or stringers and floorbeams, or by the floorbeams alone. Stringers are used for beam bridges and are commonly used for truss bridges, while the stringerless floor is commonly used on plate girder bridges. The sub-floor slabs are commonly calculated to carry the dead load due to the weight of the slab and of the wearing surface, and a live load consisting of a uniform load per square foot or a concentrated moving load. The thickness of reinforced concrete slabs in short spans is commonly determined by the concentrated moving load. The stresses in reinforced concrete slabs due to a concentrated load will depend upon the distribution of the load over the slab. The different methods for the distribution of concentrated loads in use in different specifications have been described and the specifications adopted by the author have already been given.

Design of Reinforced Concrete Floor Slabs.—The live loads and the distribution of loads on floor slabs as specified by the author are given on pages 112d and 112f. The concrete should be a 1–2–4 Portland cement concrete that will give a compressive strength of not less than 2,000 lb. per sq. in. when tested in cylinders 8 in. in diameter and 16 in. long after having been stored for 28 days in moist air. Allowable compression in slabs, 650 lb. per sq. in.; allowable tensile stress in steel, 16,000 lb. per sq. in., modulus of elasticity of steel to be taken as 15 times the modulus of elasticity of concrete, allowable shear as a measure of diagonal tension 40 lb. per sq. in.; punching shear 120 lb. per sq. in., bond stress in slabs 120 lb. per sq. in.

The thickness of floor slabs when supported on longitudinal joists or stringers is given in Table II and the thickness of floor slabs when supported on cross floorbeams (stringerless floor) is given in Table III. The reinforcing steel for reinforced concrete floor slabs is given in Table IV. The reinforcement given in the table is to be placed at the bottom of slabs calculated as simply supported and at top and bottom of slabs calculated as continuous or partially continuous.

TABLE II.

THICKNESS OF REINFORCED CONCRETE FLOOR SLABS, USED WITH JOISTS.

Simply Supported, Reinforcement on Under Side Only.							Fully Continuous, Reinforcement on Both Sides.						
Span, Ft.	12-Ton Truck.		15-Ton Truck.		20-Ton Truck.		Span, Ft.	12-Ton Truck.		15-Ton Truck.		20-Ton Truck.	
	Weight of Wearing Surface, Lb. per Sq. Ft							Weight of Wearing Surface, Lb. per Sq. Ft.					
	0	100	0	100	0	100		0	100	0	100	0	100
	in.	in.	in.	in.	in.	in.		in.	in.	in.	in.	in.	in.
2	5¼	5¼	5½	5½	5½	5½	2	4½	4½	4¾	4¾	4¾	4¾
3	5¾	6	6¼	6¼	6½	6¾	3	5	5	5¼	5¼	5½	5½
4	6¼	6½	6½	6¾	7	7½	4	5¼	5¼	5½	5¾	6	6¼
5	6½	6¾	6¾	7	7¾	8	5	5½	5¾	5¾	6	6½	6½
6	6¾	7	7½	7¾	8¼	8½	6	5¾	6	6	6¼	6¾	7

Center of reinforcing 1 in. from face of slab. Impact 30 per cent.
Reinforced as in Table IV.

TABLE III.

THICKNESS OF REINFORCED CONCRETE FLOOR SLABS, USED WITHOUT JOISTS.

Simply Supported, Reinforcement on Under Side Only.							Partially Continuous, Reinforcement on Both Sides.						
Span, Ft.	12-Ton Truck.		15-Ton Truck.		20-Ton Truck.		Span, Ft.	12-Ton Truck.		15-Ton Truck.		20-Ton Truck.	
	Weight of Wearing Surface, Lb. per Sq. Ft.							Weight of Wearing Surface, Lb. per Sq Ft.					
	0	100	0	100	0	100		0	100	0	100	0	100
	in.	in.	in.	in.	in.	in.		in.	in.	in.	in.	in.	in.
2	5¾	5¾	6	6	6½	6½	2	5¼	5¼	5½	5½	5¾	5¾
3	6¼	6¼	6½	6½	7	7	3	5¾	5¾	5¾	6	6½	6½
4	6½	6½	6¾	7	7¾	7¾	4	6	6	6¼	6¼	6¾	7
5	6¾	7	7	7½	8	8¼	5	6	6¼	6½	6½	7	7½
6	7	7½	7½	7¾	8¼	8½	6	6¼	6½	6½	7	7¾	7¾
7	7	7¾	7¾	8¼	8¾	9	7	6½	6¾	6¾	7½	8	8¼
8	7½	8¼	8¼	8¾	9¼	9¾	8	6¾	7¼	7½	8	8½	9
9	8	8¾	8¾	9¼	10	10½	9	7½	8	8	8½	9	9½
10	8½	9¼	9¼	10	10½	11¼	10	7¾	8½	8½	9	9½	10

Center of reinforcing 1 in. from face of slab for slabs less than 7½ in. thick.
Center of reinforcing 1¼ in. from face of slab for slabs 7½ in. and over, in thickness.
Impact 30 per cent. of live load.
Reinforced as in Table IV.

Examples of Reinforced Concrete Floor Slabs.—The reinforced concrete floor slabs used by the Wisconsin Highway Commission are given in Fig. 14, Fig. 15, Fig. 21 and Fig. 22. The floor slabs used by the Iowa Highway Commission are given in Fig. 12, Fig. 13, Fig. 17, and Fig. 24. For a stringerless floor the slabs used by the Iowa commission agree very closely with the values given in Table III.

TABLE IV.

REINFORCEMENT FOR REINFORCED CONCRETE FLOOR SLABS.

The reinforcement given in this table is to be used at the bottom of slabs figured as simple supported, and at the top and bottom of slabs figured as continuous or partially continuous over the supports. Longitudinal reinforcement ½ in. round or square bars spaced two feet centers.

Total Thickness, In.	Concrete Outside Center of Steel, In.	Area of Steel per Foot Width, Sq. In.	Weight of Slab, Lb. per Sq. Ft.	Spacing of Bars in Inches.							
				Round.				Square.			
				⅜ In.	½ In.	⅝ In.	¾ In.	⅜ In.	½ In.	⅝ In.	¾ In.
5	1	0.370	63	3½	6¼	10		4½	8	12½	
5½	1	0.416	69	3¼	5½	9		4	7¼	11¼	
6	1	0.462	75	2¾	5	8		3¾	6½	10	
6½	1	0.508	81	2½	4¾	7¼		3¼	6	9¼	
7	1	0.554	88	2¼	4¼	6¾		3	5½	8½	
7½	1¼	0.578	94	2¼	4	6½		3	5¼	8	
8	1¼	0.624	100	2	3¾	6		2¾	4¾	7½	
8½	1¼	0.670	106	2	3½	5½	8	2½	4½	7	10
9	1¼	0.716	113		3¼	5¼	7½		4¼	6½	9½
9½	1¼	0.762	119		3	4¾	7		4	6	9
10	1¼	0.809	125		2¾	4½	6½		3¾	5¾	8½
11	1¼	0.901	138		2½	4	6		3¼	5¼	7½
12	1¼	0.993	150			3¾	5¼		3	4¾	6¾
Interpolate for intermediate slabs.											

The Illinois Highway Commission for stringer spacings of about 2½ ft. uses a concrete sub-floor 4 in. thick, with a 4 in. concrete wearing surface, or a 3 in. creosoted timber block wearing surface. The concrete sub-floor, 4 in. thick, is reinforced on the under side with ½ in. square bars, spaced 6 in. centers and centers 1 in. above lower edge. Transverse reinforcement consists of ⅝ in. square bars spaced 12 in. centers. The concrete is specified as 1–2–3½ mix, and is designed for a stress of 800 lb. per sq. in.

The West Virginia Highway Commission specifies 1–2–4 concrete and a minimum thickness of slab of 5 in. to the center of the tension reinforcement.

The Ohio Highway Commission specifies concrete slabs for different stringer spacings as follows: 5 in. slab for 2 ft. spacing; 6 in. slab for 3 ft. spacing; 6 in. slab for 4 ft. spacing.

Specifications for highway bridges of the state of Nebraska specify slabs made of concrete of a 1–2–4 mix, 6 in. thick reinforced with ½ in. round bars spaced 6 in. centers. The bottom of the concrete to be 1 inch below top of joists.

The standard reinforced concrete floor used by the Michigan Highway Commission is shown in Fig. 8. The slab is 6½ in. thick at the center and 6 in. thick at the curb. The details of the floor are shown in the cut.

Buckle Plates.—Buckle plates are made by "dishing" flat plates as in Table 55, Part II. The width of the buckle W or length L, varies from 2 ft. 6 in. to 5 ft. 6 in. The buckles may be turned with the greater dimension in either dimension of the plate. Several buckles may be put in one plate, all of which must be of the same size and be symmetrically placed. Buckle plates are made ¼ in., $\frac{5}{16}$ in., ⅜ in. and $\frac{7}{16}$ in. thick. Buckle plates should be firmly bolted or riveted around the edges with a maximum spacing of 6 inches, and should be supported transversely between the buckles. The process of buckling distorts the plates and an extra width should be ordered, and the plate should be trimmed after the process is complete. The buckle plates are usually supported on the tops of the stringers, but may be fastened to the bottoms of the stringers. The space above the buckles is filled with concrete which carries the wearing surface. Buckle plates are now seldom used except for special floors and heavy floors where the weight of a reinforced concrete floor would be too great, or where it is necessary to cut down the clearance.

Plank Floors.—As long as an excellent grade of timber was available and the concentrated loads were not excessive, timber floors were quite satisfactory when properly constructed. Plank floors should be of white oak, long leaf yellow pine or similar timber, laid transversely. Where two layers of plank are used the lower layer is laid diagonally. Planks should be from 8 in. to 12 in. wide and not less than 3 in. thick. To carry modern auto trucks the plank should have a minimum thickness in inches of three halves the spacing of the stringers in feet. Planks should

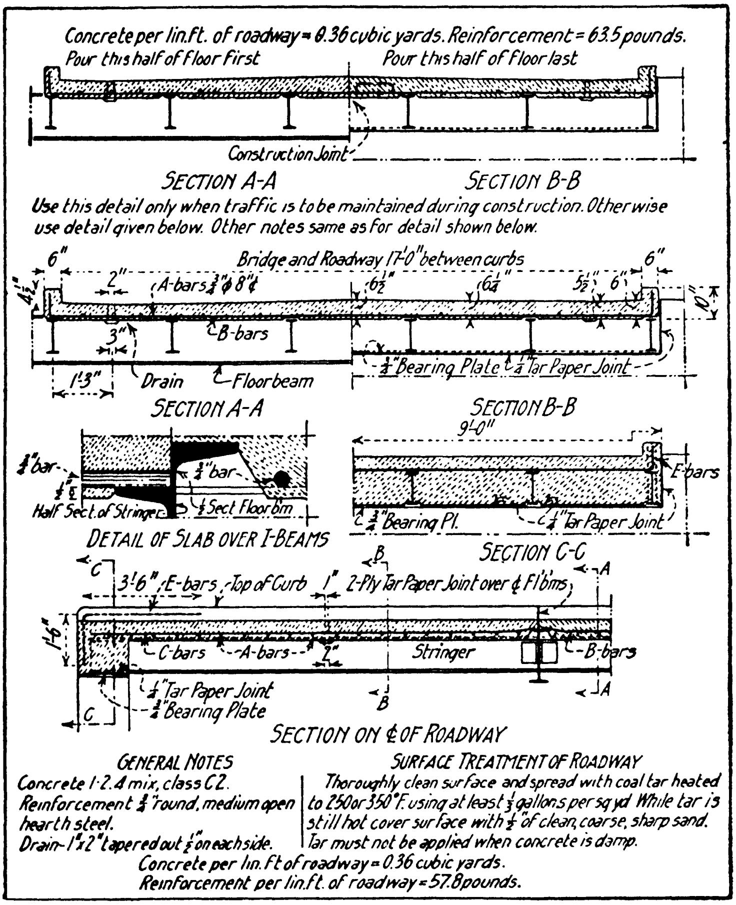


FIG. 8. REINFORCED CONCRETE FLOOR, MICHIGAN HIGHWAY COMMISSION.

be laid from ¼ in. to ½ in. apart so that water will not be retained, but will run through and will give the planks an opportunity to dry out. Where more than one layer of planks is used a liberal coating of coal tar to the upper side of the lower planks and to the lower side of the upper planks will materially prolong the life of the floor. The timber in floors made of more than one layer of planks should be creosoted. Each plank should be solidly spiked to each joist with spikes having a length not less than twice the thickness of the plank, or 6-in. spikes for 3-in. plank and 8-in. spikes for 4-in. plank. Where steel joists are used, spiking strips about 3 in. by 8 in. are bolted to the tops of all joists, or spiking strips 4 in. by 6 in. are bolted to the sides of three lines of joists

under each plank length. When the latter method is used the floor planks are fastened to the intermediate joists by bending spikes, driven through the floor plank, around the upper flanges of the joist. For specifications for plank floors, see the author's "General Specifications for Steel Highway Bridges."

The thickness of plank for different loadings and spans calculated for the allowable stresses required by the author's specifications are given in Table V.

Laminated Timber Floor.—Highway bridge floors are sometimes made by placing 2 in. by 4 in., 2 in. by 6 in., or 3 in. by 8 in. timbers on edge and spiking them together. A waterproof wearing surface is placed on top of the laminated base. The safe spans for a laminated timber floor may be taken the same as for planks 12 inches wide.

The Oregon Highway Commission uses laminated wood floors made of 3 in. by 8 in. timbers placed on edge and spiked together at intervals of not less than 18 in. "The timbers shall preferably be long enough to extend the full width of the roadway, and in no case shall more than two lengths be used in the width of roadway. Every fifth timber shall project $\frac{1}{2}$ in. above the intervening four pieces, to furnish a grip for the waterproof wearing surface."

A laminated floor made of 2 in. by 4 in. pine timbers placed on edge and spiked together was used for reflooring 23d Street Bridge, Denver, Colorado. The laminated timber base is covered with an asphalt paving $1\frac{1}{2}$ inches thick.

TABLE V.

THICKNESS OF 12-INCH FLOOR PLANK.

For 8-inch plank add 23 per cent to the thickness of plank.
Thickness in Inches, Actual Size, No Impact.

Spacing of Joists, In.	10-Ton Auto Truck.	12-Ton Auto Truck.	15-Ton Auto Truck.	20-Ton Auto Truck.
12	2	2	2	2
15	$2\frac{3}{8}$	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{5}{8}$
18	$2\frac{3}{4}$	$2\frac{7}{8}$	3	$3\frac{1}{8}$
21	3	$3\frac{1}{4}$	$3\frac{3}{8}$	$3\frac{5}{8}$
24	$3\frac{1}{4}$	$3\frac{1}{2}$	$3\frac{3}{4}$	4
27	$3\frac{1}{2}$	$3\frac{3}{4}$	4	$4\frac{1}{2}$
30	$3\frac{3}{4}$	4	$4\frac{3}{8}$	$4\frac{7}{8}$
33	4	$4\frac{1}{4}$	$4\frac{5}{8}$	$5\frac{1}{4}$
36	$4\frac{1}{4}$	$4\frac{1}{2}$	$4\frac{7}{8}$	$5\frac{1}{2}$

Allowable Stresses.—Bending stress, 1,500 lb. per sq. in.; bearing across fiber, 400 lb. per sq. in.
Minimum thickness of plank allowed by Ketchum's specifications is 3 in.; maximum spacing of joists is 30 in.

Creosoted Timber Floor.—Creosoted timber may be used as a sub-floor for a creosoted timber block wearing surface, for a bituminous wearing surface, or may carry a gravel or earth fill, or may have no wearing surface.

Specifications for Creosoted Timber.—Timber used for all creosoted floor timbers except blocks shall be first-class oak, long-leaf yellow pine or Oregon fir. It shall be cut from live trees and shall be straight grained, free from shakes, large or loose knots, decayed wood, worm holes or other defects that will impair its strength or durability. It shall be sawed straight and true and shall be full size. All timber shall be impregnated with at least 12 lb. of creosote oil per cubic foot of timber. The creosote oil shall be a pure coal-tar product free from any adulteration. It shall be free from any tar or any petroleum oil or petroleum residue. The specific gravity at 100° F. shall be at least 1.03, but not more than 1.07. The creosote oil shall comply with the specifications of the American Railway Engineering Association for creosote oil. The timber shall be impregnated with creosote oil by the full cell process. The details of the treatment shall comply with the specifications of the American Railway Engineering Association for the treatment of ties with creosote oil.

The timbers for the sub-floor shall be surfaced on one side and one edge, and shall not vary more than $\frac{1}{16}$ in. from the specified thickness. The timbers shall be laid with the surfaced side down with tight joints, and shall be fastened to the outside spiking strips with two 6-in. lag screws at each end of each plank, and to the intermediate stringers with two spikes in each stringer, the length of the spikes to be at least twice the thickness of the floor planks. The fellow guard shall be bolted to the stringers with $\frac{5}{8}$-in. bolts spaced not more than 5 ft. centers.

WEARING SURFACES FOR HIGHWAY BRIDGE FLOORS.—The wearing surface of a highway bridge floor should satisfy the usual conditions for a pavement and in addition should not have an excessive weight; as an increase in dead load on the bridge increases the necessary amount of steel in the floor supports and the trusses and increases the total cost. The most common wearing surfaces will be briefly described.

Concrete.—A concrete wearing surface is laid on top of the concrete slab by the Illinois Highway Commission as follows:—The wearing surface shall have a thickness of not less than 4 inches. The lower 2 in. of the wearing surface shall be made of concrete mixed in the proportions of one part Portland cement, 2 parts clean sand and 4 parts clean gravel or broken stone that will pass a $1\frac{1}{2}$-in. ring. The concrete shall be thoroughly mixed in a batch mixer to a jelly-like consistency and shall be placed immediately on the sub-floor slab. Upon this concrete layer shall be immediately laid a 2-in. layer of mortar made by mixing one part Portland cement and 2 parts of clean, coarse sand. The mortar shall be mixed to a jelly-like consistency in a batch mixer and shall be immediately placed upon the freshly laid concrete. Before the mortar has begun to set it shall be finished off with a wood float, and before it has hardened it shall be roughened by brushing with a stiff vegetable brush or broom.

The concrete slab and the concrete wearing surface are commonly laid in one operation, the wearing surface being finished up as for a concrete pavement.

Creosoted Timber Blocks.—The blocks shall be made of prime sound long-leaf yellow pine or Oregon fir and shall contain no loose knots, worm holes or other defects, and shall be well manufactured. No wood averaging less than 6 rings to the inch, measured radially from the center of the heart shall be used. The blocks shall have a depth as specified, but the depth shall not be less than 3 in. The blocks shall be from 6 to 10. in. long. The width shall be from 3 to 4 in., but the blocks in any contract shall have the same width. A variation of $\frac{1}{16}$ in. in depth and $\frac{1}{8}$ inch in width will be permitted. The width shall be greater or less then the depth by not less than $\frac{1}{4}$ in. The blocks shall be impregnated with creosote oil by the full cell process. The creosote oil and the method of creosoting timber blocks shall be the same as specified for creosoted timber. All creosoted timber blocks shall contain not less than 16 lb. of creosote oil per cubic foot of timber.

Laying Creosoted Timber Blocks.—When the creosoted timber blocks are laid on a creosoted timber base, a layer of tar paper shall be laid on the timber base. When creosoted timber blocks are laid on a concrete floor slab, a layer of dry cement mortar made by mixing dry one part of Portland cement and four parts of clean dry sand shall be spread on the dry floor slab. The cement cushion shall be rolled to a thickness of $\frac{1}{2}$ in. As the blocks are laid on the concrete slab the sand and cement shall be moistened by sprinkling and the blocks shall be laid before the cement has had time to set. The blocks shall be laid at right angles to the length of the bridge in parallel lines, with the grain vertical. The blocks shall break joints at least 3 in. Two lines of blocks shall be laid next to the curb with the long dimension of the block parallel to the bridge, and the remainder of the blocks shall be laid at right angles to those blocks. The blocks shall be laid with open joints, $\frac{1}{8}$-in. open joints transversely, $\frac{1}{4}$-in. open joints longitudinally. Expansion joints not less than 1 in. thick the full depth of the block shall be provided along each curb, and transverse joints not less than $\frac{1}{2}$ in. thick shall be provided every 50 ft. in length of the bridge. These joints shall be kept closed until the blocks are all laid, and the space is then to be filled with a bituminous filler. After the blocks have been laid they shall be tamped or rolled to firm bearing. All defective, broken, damaged or displaced blocks shall be removed and replaced with sound blocks. All joints and expansion joints shall then be filled to a depth of two-thirds the depth of the block with a satisfactory bituminous filler. The filler shall not be brittle at 0° F. nor flow at 120° F. The filler shall be applied at a temperature of not less than 300° F. After the first application has set the joints shall be filled to the proper height with a second coat. Joints shall be filled only in dry weather, when the temperature is not less than 50° F. Before the second coat has hardened a layer of sand $\frac{1}{4}$ in. thick shall be spread on the surface and shall be swept into the joints.

Bituminous Wearing Surface Floors.—Bituminous wearing surface floors may be laid on a creosoted timber sub-floor or on a concrete sub-floor.

Bituminous Wearing Surface on Timber Sub-Floor.—**The bituminous wearing surface may be put on hot by the standard method, or by a cold process. The specifications adopted in 1917 by the Illinois Highway Commission are as follows:**

Bituminous Wearing Surface—Hot Penetration Method. Illinois Highway Commission.

Asphalt.—The asphalt used for bituminous wearing surface shall conform to the following requirements: Asphalt shall have a specific gravity at 25° C. of not less than 0.97 nor more than unity. It shall be soluble in cold carbon disulphide to the extent of at least 98 per cent. Of the total bitumen, not less than 22 per cent nor more than 30 per cent shall be insoluble in 86° B. naphtha. When 20 grams (in a tin dish $2\frac{1}{2}$ in. in diameter and $\frac{3}{4}$ in. deep with vertical sides) are maintained at a temperature of 163° C. for 5 hours in a N. Y. testing laboratory oven, the evaporation loss shall not exceed 2 per cent and the penetration shall not have been decreased more than 25 per cent. The fixed carbon shall not exceed 16 per cent by weight. The penetration as determined with the Dow machine using a No. 2 needle, 100 g. weight, 5 seconds time, and a temperature of 25° C. shall be not less than 30 nor more than 50. The asphalt shall contain not to exceed 6 per cent by weight of paraffine scale.

Aggregate.—The aggregate shall consist of screened gravel, which shall have been approved by the engineer, dry, free from dust, dirt and clay, and graded in size from $\frac{3}{8}$ in. to $\frac{3}{4}$ in.

Cleaning Sub-Planking.—Before placing the wearing surface, the sub-planking shall be thoroughly cleaned from all foreign material and the cracks shall be filled and the plank covered to a depth of approximately $\frac{1}{8}$ in. with asphalt of the character herein specified, which shall be applied at a temperature of not less than 400° F. The sub-planking shall be dry when the asphalt is applied.

Placing Wearing Surface.—The gravel shall be spread on the asphalt covering while the same is hot and in a quantity which will just cover the asphalt. The thickness must not exceed that which will be formed by a single layer of the gravel pebbles.

Upon the material thus spread, there shall be poured hot asphalt until the interstices are all filled, the asphalt being at a temperature of not less than 400° F.

Upon the layer of asphalt thus poured there shall be spread a second layer of gravel which shall not exceed the thickness of a single layer of pebbles, but which must be spread in sufficient quantity to cover completely the layer of asphalt.

Upon the layer of gravel thus spread there shall be poured hot asphalt until all the interstices are filled, the asphalt having a temperature of not less than 400° F.

Finish.—The surface shall then be covered with a layer of pebbles just sufficient to cover the asphalt, the pebbles to be well rolled or tamped into the asphalt and the surface finally covered with coarse sand sufficient to take up any free asphalt. After the surface has stood for one day, it may be opened to traffic.

Bituminous Wearing Surface—Cold Mixing Method, using an Asphalt Emulsion. Illinois Highway Commission.

Asphalt Emulsion.—The emulsion shall consist of asphalt, water and fatty or resin soap thoroughly emulsified. It shall conform to the following requirements:

Total bitumen	Not less than 60.0 per cent
Specific gravity of dehydrated material	Not less than 1.000
Penetration of dehydrated material, 25° C., 100 gm., 5 sec.	150 to 200

Total Bitumen.—The total bitumen shall be considered as being 100 minus the sum of the percentages of water, of fatty or resin acids, of organic matter insoluble in carbon disulphide other than fatty or resin acids from the soap, or mineral matter (ash), and of ammonia.

For percentages of water, fatty or resin acids, organic matter insoluble in carbon disulphide, mineral matter (ash), and ammonia, see United States Department of Agriculture Bulletin 314, p. 41.

Specific Gravity.—Standardized pycnometers, United States Department of Agriculture Bulletin 314, p. 4.

Penetration.—A. S. T. M. Stand. Test D 5–16.

Aggregate.—The aggregate shall consist of crushed stone chips uniformly graded from $\frac{3}{4}$ in. down to dust with all dust removed, to which shall be added sufficient sand to fill all remaining voids, but not to exceed 20 per cent of the volume of the aggregate.

Cleaning Sub-Planking.—Before placing the wearing surface, the sub-planking shall be thoroughly cleaned from all foreign material and all cracks shall be filled with wood strips or oakum.

Mixing Materials.—The aggregate and the asphalt emulsion shall be mixed cold in the proportions of 1 gal. of emulsion to 1 cu. ft. of aggregate. To facilitate mixing, water to the extent of 20 per cent may be added to the emulsion. The proportions given above for mixing the aggregate and the emulsion are based on the undiluted emulsion. The mixing shall be done on a tight mixing board or in a batch concrete mixer, and shall continue until all particles of the aggregate are thoroughly coated.

Placing Wearing Surface.—After mixing, the material shall be spread upon the roadway in sufficient quantity to provide a thickness of ¾ in., after rolling or tamping.

Finish.—After the material has been rolled or tamped smooth and to a uniform thickness of ¾ in., the surface shall be given a paint coat of the emulsion applied at the rate of ¼ gal. per sq. yd., and then shall be covered with coarse sand sufficient to take up any free asphalt and to fill all voids in the surface. After the surface has stood for one day, it may be opened to traffic.

Bituminous Pavement on Concrete.—A bituminous wearing surface may be laid as on the creosoted plank sub-floor, or the wearing surface may be laid according to the following standard method. The concrete shall be dry and thoroughly clean. A bituminous wearing surface two inches thick is applied as follows: The aggregate consists of broken stone or gravel passing a one-inch screen with the dust screened out to which is added sand equal to about one-quarter to one-half the volume of the stone. The aggregates shall be heated and mixed with the bituminous material in a mechanical mixer or by hand with hot shovels. The asphalt shall be mixed not less than 20 gallons to the cubic yard of aggregate at a temperature of 350° to 400° F. The mixture shall be applied hot to the concrete surface and shall be raked with hot hoes or rakes and is rolled with a roller weighing not less than 5 tons. After the surface has been rolled a layer of hot asphalt shall be applied and a layer of coarse sand rolled into hot asphalt.

Examples of Highway Bridge Floors.—The following examples of highway bridge floors specified by different highway commissions are of interest.

The Illinois Highway Commission uses the following standard floors: (1) A reinforced concrete sub-floor 4 in. thick, and a concrete wearing surface 4 in. thick, weight 100 lb. per sq. ft.; (2) a reinforced concrete sub-floor 4 in. thick and a creosoted timber block wearing surface 3 in. thick, weight 65 lb. per sq. ft.; (3) a creosoted plank sub-floor 3 in. thick and a wearing surface of creosoted timber blocks 3 in. thick, weight 32 lb. per sq. ft.; and (4) a creosoted timber ship lap floor 3 in. thick and a wearing surface of creosoted timber blocks 3 in. thick, weight 26 lb. per sq. ft.

The Michigan Highway Commission uses the following surface treatment on concrete floor slabs. The surface of the concrete is thoroughly cleaned and ⅓ of a gallon per sq. yd. of coal tar heated to a temperature of 250° to 350° F. is spread over the slab. While the tar is hot the surface is evenly covered with a layer ½ in. thick of clean, sharp, coarse sand.

The Wisconsin Highway Commission does not specify a wearing coat on top of concrete floor slabs.

The Iowa Highway Commission uses either a 3 in. fill of gravel or a creosoted block floor 3 in. thick. Concrete slabs are covered with a bituminous coating made by applying ⅓ of a gallon per sq. yd. of hot tar to the clean dry slab. A layer of coarse dry sand is heated and sifted on top of the tar.

Cost of Floors.—The costs of highway bridge floors were estimated by Mr. Clifford Older, bridge engineer, Illinois Highway Commission in 1915 as follows: Concrete in sub-floors including reinforcing steel, $12.00 per cu. yd.; concrete wearing surface, 4 in. thick, $0.90 per sq. yd.; creosoted sub-plank (12-lb. treatment) in place, $70 per thousand feet B. M.; creosoted blocks 3 in. thick, in place, $1.80 per sq. yd.; bituminous gravel wearing surface, ¾ in. thick, $0.60 per sq. yd. The weights and costs of the Illinois Highway Commission standard floors were as follows: concrete sub-floor 4 in. thick and concrete wearing surface 4 in. thick, weighs 100 lb. per sq. ft., and costs $2.95 per sq. yd.; concrete sub-floor 4 in. thick, and creosoted blocks 3 in. thick, weighs 65 lb. per sq. ft., and costs $3.25 per sq. yd.; creosoted plank sub-floor 3 in. thick, and creosoted blocks 3 in. thick, weighs 32 lb. per sq. ft., and costs $4.10 per sq. yd.; creosoted plank sub-floor 3 in. thick, and bituminous wearing surface ¾ in. thick, weighs 26 lb. per sq. ft., and costs $3.00 per sq. yd.

DESIGN OF STRINGERS.—Stringers or joists support the floor and in turn are supported by the floorbeams. The joists may be supported on the tops of the floorbeams or may be framed into the floorbeam by the use of connection angles. Where concrete floors are used the steel joists should either be supported on the tops of the floorbeams or if framed into the floorbeams should have the upper flanges of the beams coped so that the tops of the joists will be on the same level as the floorbeams. The loads carried by the joists are (1) the dead load which is made up of the weight of the joists, the floor slab and the wearing surface; (2) a uniform live load, or a concentrated moving load. The uniform live load and the concentrated moving loads are the same as the loads used in designing the floor slabs, but the distribution of the concentrated load is not the same.

The distribution of the moving concentrated load to the joists as specified by different highway commissions and others, and by the author have already been given.

Steel Stringers.—The sizes of steel I-beams of minimum weights required for stringers with different spacings to carry a dead load of 100 lb. per sq. ft. and a 20-ton auto truck with 30 per cent impact or a live load of 125 lb. per sq. ft. with 30 per cent impact are given in Fig. 9; and to carry a dead load of 100 lb. per sq. ft. and a 15-ton auto truck with 30 per cent impact or a live load of 100 lb. per sq. ft. with 30 per cent impact are given in Fig. 10. The sizes of steel I-beams of minimum weights required to carry a dead load of 100 lb. per sq. ft. and a 15-ton auto truck without impact or a live load of 100 lb. per sq. ft. without impact are given in Fig. 11. The steel stringers used by the Wisconsin Highway Commission to carry a 15-ton road roller without impact, and the steel stringers used by the Iowa Highway Commission to carry a 15-ton traction engine without impact are practically the same as those given in Fig. 11.

Timber Joists.—The sizes of timber stringers or joists for different spacings and spans to carry a 20-ton auto truck are given in Table VI; to carry a 15-ton auto truck in Table VII, and to carry a 10-ton auto truck in Table VIII. The timber joists were designed for the following unit stresses, to be used without impact: Allowable bending stress, 1,500 lb. per sq. in.; allowable bearing across the grain, 400 lb. per sq. in.; allowable longitudinal shear in beams, 140 lb. per sq. in. The maximum spacings of timber joists for short spans are determined by the longitudinal shear.

TABLE VI.

SPACING OF TIMBER STRINGERS OR JOISTS.

Calculated for 20-ton Auto Truck, Without Impact.

Nominal Size of Joists, In.	Maximum Spacing in Feet for Different Spans in Feet.							
	6	8	10	12	14	16	18	20
3 × 10......	0.7	0.7	0.6					
4 × 10......	0.9	0.9	0.8					
3 × 12......	0.8	0.8	0.8	0.7				
4 × 12......	1.1	1.1	1.1	1.0				
3 × 14......	1.0	1.0	1.0	1.0	0.8			
4 × 14......	1.3	1.3	1.3	1.3	1.1	1.0		
6 × 14......	2.0	2.0	2.0	2.0	1.7	1.5	1.3	1.2
4 × 16......	1.5	1.5	1.5	1.5	1.5	1.3	1.2	1.0
6 × 16......	2.2	2.2	2.2	2.2	2.2	2.0	1.7	1.5

The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists in feet divided by four feet.

Joists were designed for allowable stresses as follows: Cross-bending, 1,500 lb. per sq. in.; bearing across the grain 400 lb. per sq. in.; longitudinal shear 140 lb. per sq. in.

Spacing of joists for spans to left of heavy line are determined by longitudinal shear.

DESIGN OF FLOORBEAMS.—The floor loads may be carried to the floorbeams by means of stringers or joists, or the loads may be carried to the floorbeams directly by the floor slabs. The loads carried by the floorbeams consist of (1) the dead load which is the weight of the floor system; (2) a uniform live load; or a concentrated moving load. The uniform live loads are the same as the uniform live loads used in designing the floor slabs and stringers, but the distribution of the concentrated moving load is not the same as for either the floor slabs or the stringers. The distribution of the moving concentrated load to floorbeams as specified by different highway commissions and others, and by the author have already been given.

TABLE VII.

Spacing of Timber Stringers or Joists.

Calculated for 15-ton Auto Truck, Without Impact.

Nominal Size of Joists, In.	Maximum Spacing in Feet for Different Spans in Feet.							
	6	8	10	12	14	16	18	20
3 × 10.....	1.0	1.0	0.8					
4 × 10.....	1.3	1.3	1.1	0.9				
3 × 12......	1.1	1.1	1.1	1.0				
4 × 12......	1.6	1.6	1.6	1.4	1.2	1.0		
3 × 14......	1.4	1.4	1.4	1.4	1.2	1.0		
4 × 14......	1.9	1.9	1.9	1.9	1.6	1.4	1.2	1.1
6 × 14.....	2.8	2.8	2.8	2.8	2.4	2.0	1.8	1.6
4 × 16......	2.1	2.1	2.1	2.1	2.1	1.8	1.6	1.5
6 × 16......	3.1	3.1	3.1	3.1	3.1	2.7	2.4	2.2

The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists in feet divided by four feet.

Joists were designed for allowable stresses as follows: Cross-bending, 1,500 lb. per sq. in.; bearing across the grain, 400 lb. per sq. in.; longitudinal shear, 140 lb. per sq. in.

Spacing of joists for spans to left of heavy line are determined by longitudinal shear.

TABLE VIII.

Spacing of Timber Stringers or Joists.

Calculated for 10-ton Auto Truck, Without Impact.

Nominal Size of Joists In.	Maximum Spacing in Feet for Different Spans in Feet.							
	6	8	10	12	14	16	18	20
3 × 10......	1.4	1.4	1.2	1.0	0.9			
4 × 10......	2.0	2.0	1.7	1.4	1.2	1.0		
3 × 12......	1.8	1.8	1.8	1.5	1.3	1.1	1.0	
4 × 12......	2.4	2.4	2.4	2.0	1.8	1.5	1.4	1.2
3 × 14......	2.0	2.0	2.0	2.0	1.8	1.5	1.4	1.2
4 × 14......	2.8	2.8	2.8	2.8	2.4	2.1	1.9	1.7
6 × 14......	4.1	4.1	4.1	4.1	3.5	3.1	2.8	2.5
4 × 16......	3.2	3.2	3.2	3.2	3.2	2.8	2.5	2.2
6 × 16......	4.7	4.7	4.7	4.7	4.7	4.1	3.6	3.3

The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists in feet divided by four feet.

Joists were designed for allowable stresses as follows: Cross-bending, 1,500 lb. per sq. in.; bearing across the grain, 400 lb. per sq. in.; longitudinal shear, 140 lb. per sq. in.

Spacing of joists for spans to left of heavy line are determined by longitudinal shear.

Steel I-Beam Floorbeams.—The sizes of steel I-beams required for floorbeams for panel lengths of 10 ft. to 24 ft. and widths center to center of trusses or girders of 15 ft. to 26 ft. to carry a dead load of 100 lb. per sq. ft., and a 20-ton auto truck with 30 per cent impact, or a uniform live load of 125 lb. per sq. ft. with 30 per cent impact are given in Fig. 9; while the floorbeams required to carry a 15-ton auto truck with 30 per cent impact, or a uniform live load of 100 lb. per sq. ft. with 30 per cent impact are given in Fig. 10. It will be noted that the uniform live load controls for wide roadways or for long panels.

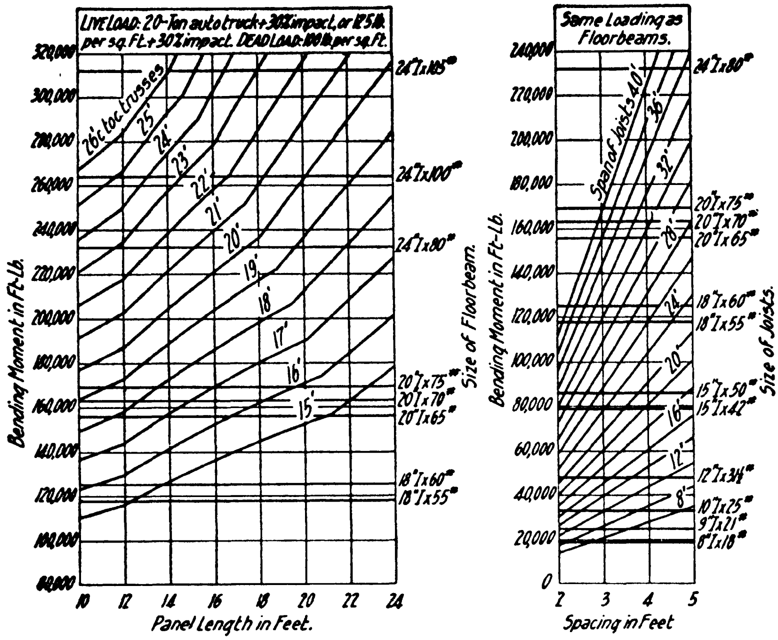


FIG. 9. BENDING MOMENTS IN FLOORBEAMS AND STRINGERS FOR 20-TON AUTO TRUCK. (30 PER CENT IMPACT). CONCRETE FLOOR.

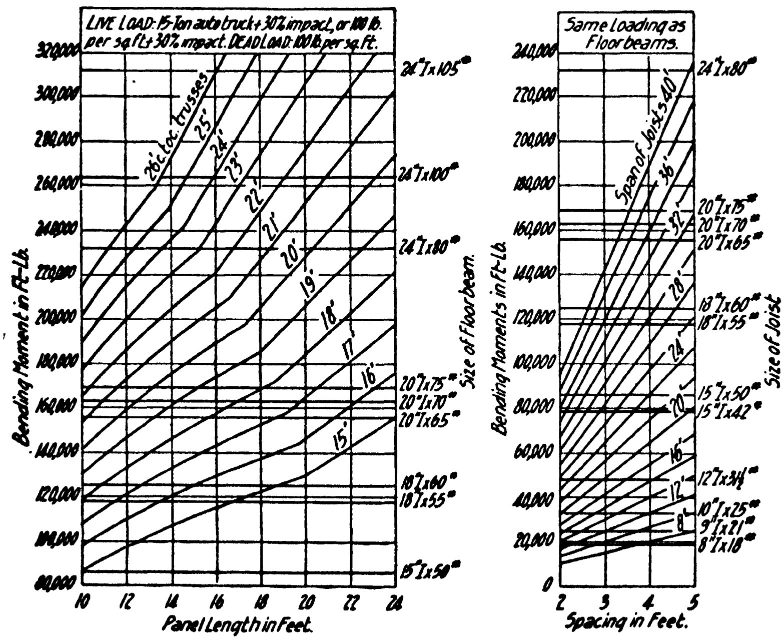


FIG. 10. BENDING MOMENTS IN FLOORBEAMS AND STRINGERS FOR 15-TON AUTO TRUCK. (30 PER CENT IMPACT). CONCRETE FLOOR.

For a bridge 17 ft. center of trusses and 18 ft. panels, from Fig. 9 the required floorbeam is a 24 in. I @ 80 lb., while from Fig. 10 the required floorbeam is a 20 in. I @ 65 lb.

The sizes of steel I-beams required for floorbeams for panel lengths of 10 ft. to 24 ft., and widths center to center of trusses or girders of 15 ft. to 26 ft. to carry a dead load of 100 lb. per sq. ft. and a 15-ton auto truck without impact, or a uniform live load of 100 lb. per sq. ft. without impact are given in Fig. 11. These are practically the floorbeams required by the specifications of the Illinois, Iowa, and Wisconsin Highway Commissions. Steel stringers for the same loading are given in Fig. 11.

The bending moments for the design of built-up floorbeams may be obtained from Fig. 9, Fig. 10, or Fig. 11.

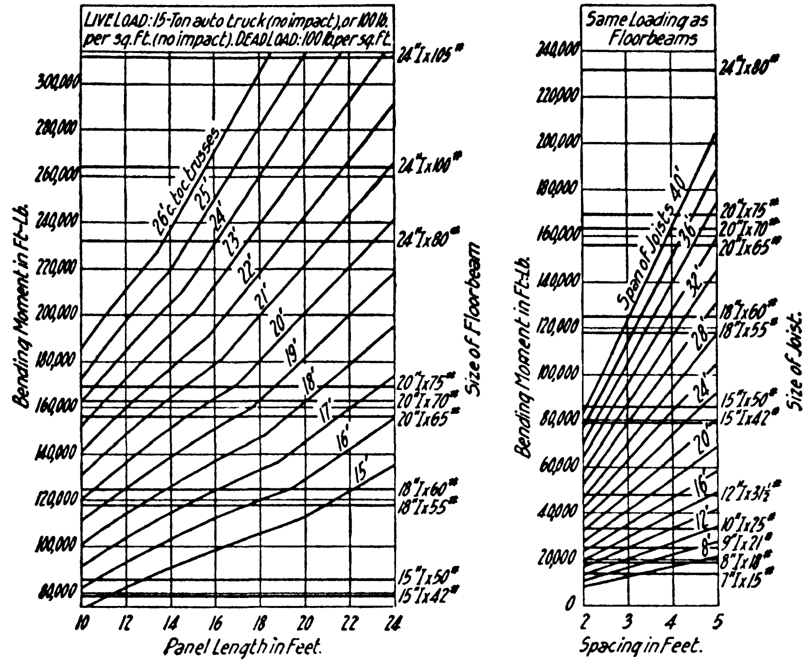


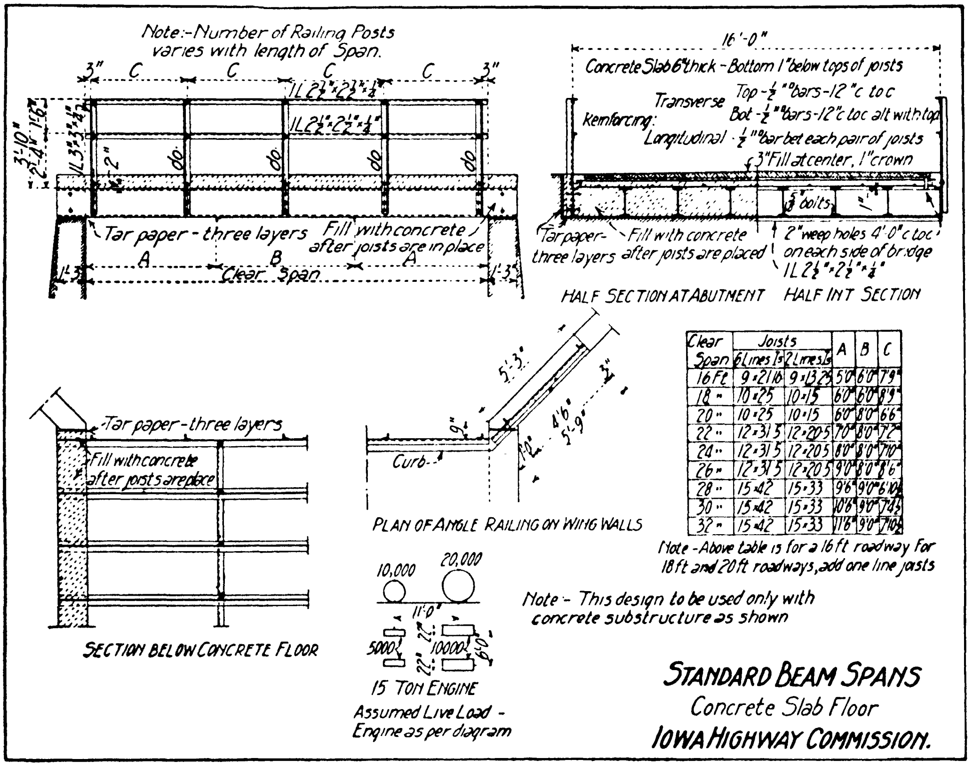
FIG. 11. BENDING MOMENTS IN FLOORBEAMS AND STRINGERS FOR 15-TON AUTO TRUCK. (NO IMPACT.) CONCRETE FLOOR.

CALCULATION OF STRESSES.—For the calculation of the stresses in highway bridges. see the author's "The Design of Highway Bridges," also see Chapter XVI.

ALLOWABLE STRESSES.—For allowable stresses to be used in the design of steel highway bridges, see "General Specifications for Steel Highway Bridges," printed in the last part of this chapter.

SHORT-SPAN STEEL HIGHWAY BRIDGES.—The term short-span highway bridges will be assumed to include beam, low truss and plate girder bridges.

BEAM BRIDGES.—Beam bridges are made by placing steel I-beams side by side with the ends resting on the abutments. The roadway floor may be made of planks laid transversely on the tops of the beams, or of reinforced concrete. The spacing of the beams depends upon the load to be carried and upon the thickness of the floor planks or floor slabs and varies from 2 to 4 ft. Timber joists should not be spaced more than $2\frac{1}{2}$ ft. centers. A common rule for the thickness of oak floor planks is that the plank shall have at least one and one-half inch in thickness for each foot of spacing of the joists or stringers. The outside beams should be the same size as the intermediate beams. It is commonly specified that rolled beams shall have a depth not less than $\frac{1}{30}$ the span.

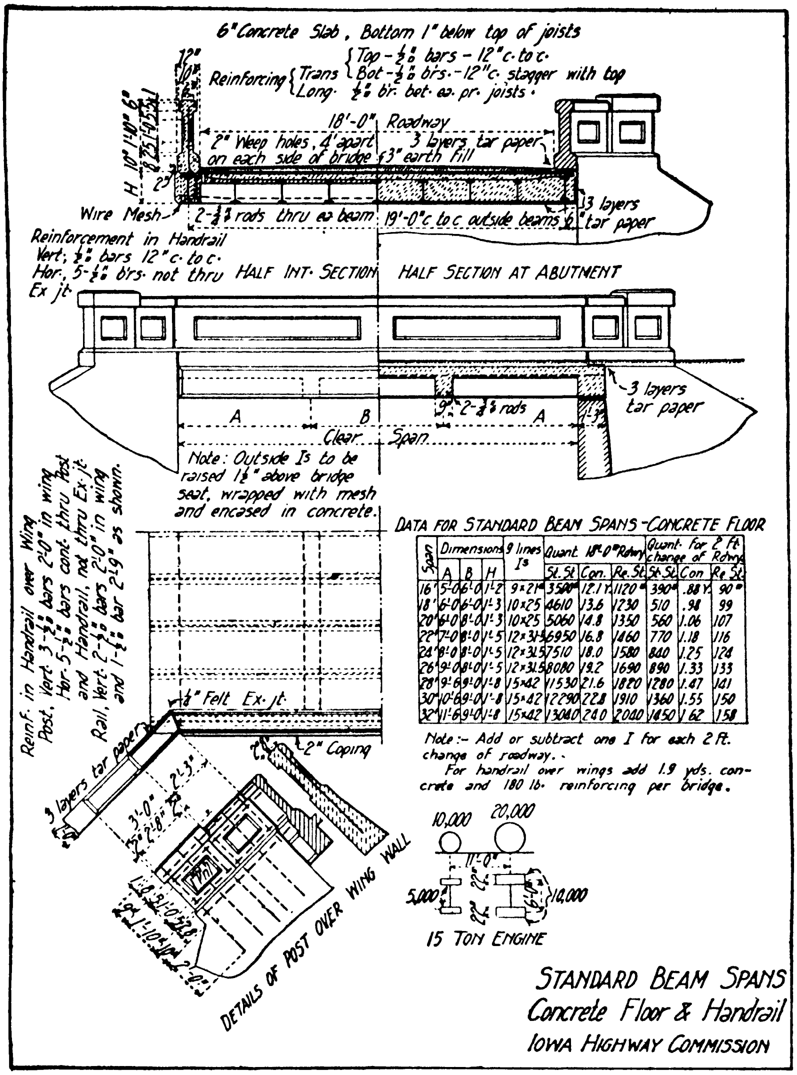


Clear Span	Joists		A	B	C
	6 Lines I's	2 Lines I's			
16 Ft	9×21½	9×13¼	5'0"	6'0"	7'9"
18 "	10×25	10×15	6'0"	6'0"	8'9"
20 "	10×25	10×15	6'0"	8'0"	6'6"
22 "	12×31.5	12×20.5	7'0"	8'0"	7'2"
24 "	12×31.5	12×20.5	8'0"	8'0"	7'10"
26 "	12×31.5	12×20.5	9'0"	8'0"	8'6"
28 "	15×42	15×33	9'6"	9'0"	6'10"
30 "	15×42	15×33	10'6"	9'0"	7'4½"
32 "	15×42	15×33	11'6"	9'0"	7'10"

FIG. 12. BEAM BRIDGES.

Standard steel beam bridges with concrete floor as designed by the Iowa Highway Commission are given in Fig. 12 and Fig. 13. The spans vary from 16 ft. to 32 ft. The details are shown in the cuts. Quantities for beam bridges with angle fence as shown in Fig. 12 are given in Table IX.

A standard steel beam bridge as designed by the Wisconsin Highway Commission is shown in Fig. 14. Data and quantities for beam spans from 10 ft. to 38 ft. are shown in Table X.



Span	Dimensions A	B	H	9 lines Is	Quant. 18'-0" Rdwy St. St	Con.	Re. St	Quant. for 2 Ft. change of Rdwy St. St	Con	Re. St
16'	5'-0	6'-0	1'-2	9×21#	3590#	12.1 Y.	1120#	390#	.88 Y.	90#
18'	6'-0	6'-0	1'-3	10×25	4610	13.6	1230	510	.98	99
20'	6'-0	8'-0	1'-3	10×25	5060	14.8	1350	560	1.06	107
22'	7'-0	8'-0	1'-5	12×31.5	6950	16.8	1460	770	1.18	116
24'	8'-0	8'-0	1'-5	12×31.5	7510	18.0	1580	840	1.25	124
26'	9'-0	8'-0	1'-5	12×31.5	8080	19.2	1690	890	1.33	133
28'	9'-6	9'-0	1'-8	15×42	11530	21.6	1820	1280	1.47	141
30'	10'-6	9'-0	1'-8	15×42	12290	22.8	1910	1360	1.55	150
32'	11'-6	9'-0	1'-8	15×42	13040	24.0	2000	1450	1.62	158

FIG. 13. BEAM BRIDGES.

The minimum sizes of I-beams for different loadings and for different spacings and spans and with a concrete and a plank floor have been calculated by the author and are given in Table XI and Table XII.

Floor planks may be spiked to spiking strips on the tops of the beams, or to spiking strips bolted on the sides of the I-beams. The floor planks are spiked to these spiking strips, and are fastened to the other beams by clinching spikes, which have been driven through the planks, around the top flanges of the beams.

The maximum span for beam bridges should be 30 ft. Riveted truss bridges or plate girders should be used for spans of 30 ft. and upwards for country bridges, and plate girders for heavy city bridges. Riveted bridges for spans of, say 40 ft., are more economical than plate girder bridges and will give fully as great a length of service if properly designed and constructed. The ends of beam bridges should always be supported on masonry abutments.

TABLE IX.

ESTIMATED QUANTITIES FOR STANDARD BEAM SPANS. IOWA HIGHWAY COMMISSION.

Span, Ft.	Structural Steel. Roadway.			Reinforced Concrete Floor. 16 Ft. Roadway.		18 Ft. Roadway.		20 Ft. Roadway.	
	16 Ft.	18 Ft.	20 Ft.	Concrete.	Steel.	Concrete.	Steel.	Concrete.	Steel.
	lb.	lb.	lb.	cu. yd.	lb.	cu. yd.	lb.	cu. yd.	lb.
16	3,370	3,780	3,800	5.6	600	6.3	680	7.0	740
18	4,280	4,810	4,820	6.2	670	7.0	750	7.7	820
20	4,720	5,300	5,320	6.8	730	7.6	830	8.5	900
22	6,340	7,130	7,150	7.4	800	8.3	900	9.2	990
24	6,840	7,690	7,710	8.0	870	9.0	980	10.0	1,070
26	7,330	8,240	8,260	8.6	930	9.7	1,050	10.7	1,150
28	10,570	11,870	11,880	9.2	1,000	10.4	1,120	11.5	1,230
30	11,240	12,620	12,640	9.8	1,060	11.0	1,200	12.2	1,310
32	11,910	13,370	13,390	10.4	1,130	11.7	1,270	13.0	1,390

Standard angle railing for wing walls as shown in Fig. 12.
Rails ∠s $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}'' \times 5'$-$9''$. Top of rail $3'$-$2''$ above grade. Post ∠s $3'' \times 3'' \times \frac{1}{4}'' \times 4'$-$3''$.
Weight of rails and posts for one wing = 90 lb.

TABLE X.

STEEL I-BEAM BRIDGES. WISCONSIN HIGHWAY COMMISSION.
Channels on outside. Weight includes railing.

Span, Ft.	16 Feet Roadway. No. Beams and Channels.	Size I-Beams, In. Lb.	Weight Structural Steel, Lb.	18 Ft. Roadway. No. Beams and Channels.	Size I-Beams, In. Lb.	Weight Structural Steel, Lb.	20 Ft. Roadway. No. Beams and Channels.	Size I-Beams, In. Lb.	Weight Structural Steel, Lb.
10	8	8—18	1,900	9	8—18	2,120	10	8—18	2,335
12	8	8—18	2,200	9	8—18	2,450	10	8—18	2,700
14	8	9—21	2,800	9	9—21	3,130	10	9—21	3,465
16	8	9—21	3,185	9	9—21	3,560	10	9—21	3,930
18	8	10—25	4,030	9	10—25	4,505	10	10—25	5,000
20	7	12—31½	4,810	8	12—31½	5,600	9	12—31½	6,285
22	8	12—31½	6,050	9	12—31½	6,790	10	12—31½	7,545
24	8	12—31½	6,435	9	12—31½	7,350	10	12—31½	8,160
26	7	15—42	8,275	8	15—42	9,420	9	15—42	10,570
28	8	15—42	10,045	9	15—42	11,275	10	15—42	12,510
30	8	15—42	10,715	9	15—42	12,025	10	15—42	13,350
32	7	18—55	12,050	8	18—55	13,930	9	18—55	15,750
34	7	18—55	12,825	8	18—55	15,760	9	18—55	16,685
36	8	18—55	15,530	9	18—55	17,570	10	18—55	19,615
38	8	18—55	16,350	9	18—55	18,405	10	18—55	20,655

	16-ft. Rdwy.	18-ft. Rdwy.	20-ft. Rdwy.
Weight in lb. of reinforcing per lineal foot....	40	44	48
Cu. yd. concrete per lineal foot..............	0.32	0.36	0.40

TABLE XI.

DEPTH IN INCHES OF I-BEAMS FOR DIFFERENT SPACINGS AND SPANS REQUIRED TO CARRY 20-TON, 15-TON AND 10-TON AUTO TRUCKS AND 30 PER CENT IMPACT. DEAD LOAD 100 LB. PER SQ. FT. MINIMUM WEIGHTS OF I-BEAMS ARE USED.

Concrete Floor.									
Span, Ft.	20-Ton Auto Truck.			15-Ton Auto Truck.			10-Ton Auto Truck.		
	Spacing, Ft.			Spacing, Ft.			Spacing, Ft.		
	2	3	4	2	3	4	2	3	4
10	8	10	12	7	9	10	6	8	9
12	9	10	12	8	9	10	7	8	9
14	10	12	15	9	10	12	8	9	10
16	10	12	15	9	12	12	8	10	12
18	12	15	15	10	12	15	9	10	12
20	12	15	18	10	15	15	9	12	12
22	12	15	18	12	15	15	10	12	15
24	15	15	18	12	15	18	10	12	15
26	15	18	18	15	15	18	12	15	15
28	15	18	20	15	18	18	12	15	18
30	15	18	20	15	18	20	12	15	18

The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists divided by six feet when reinforced concrete floor is used.
The outside beams to be the same as the intermediate beams.

TABLE XII.

DEPTH IN INCHES OF I-BEAMS FOR DIFFERENT SPACINGS AND SPANS REQUIRED TO CARRY 20-TON, 15-TON AND 10-TON AUTO TRUCKS AND 30 PER CENT IMPACT. MINIMUM WEIGHTS OF I-BEAMS ARE USED.

Plank Floor.									
Span, Ft.	20-Ton Auto Truck.			15-Ton Auto Truck.			10-Ton Auto Truck.		
	Spacing, Ft.			Spacing, Ft.			Spacing, Ft.		
	1½	2	2½	1½	2	2½	1½	2	2½
10	8	9	10	7	8	9	6	7	7
12	9	10	10	8	9	9	7	7	8
14	9	10	12	8	9	10	7	8	9
16	10	12	12	9	10	12	8	8	9
18	10	12	15	9	10	12	8	9	10
20	12	12	15	10	12	12	9	9	10
22	12	15	15	10	12	15	9	10	12
24	12	15	15	12	12	15	9	10	12
26	15	15	18	12	15	15	10	12	12
28	15	15	18	12	15	15	12	12	15
30	15	18	18	12	15	18	12	12	15

The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists divided by four feet when timber floor is used.
The outside beams to be the same as the intermediate beams.

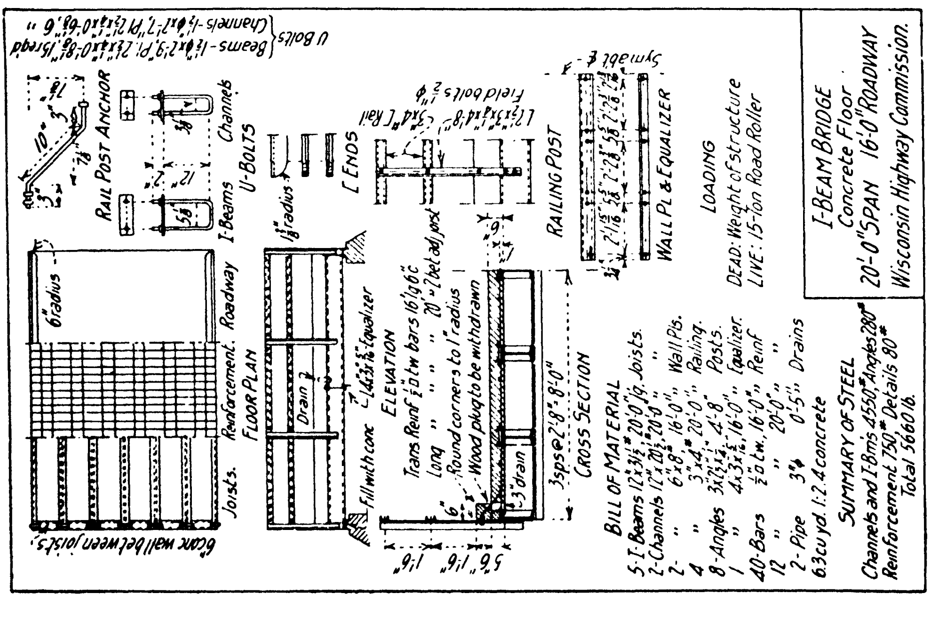


FIG. 14. BEAM BRIDGE.

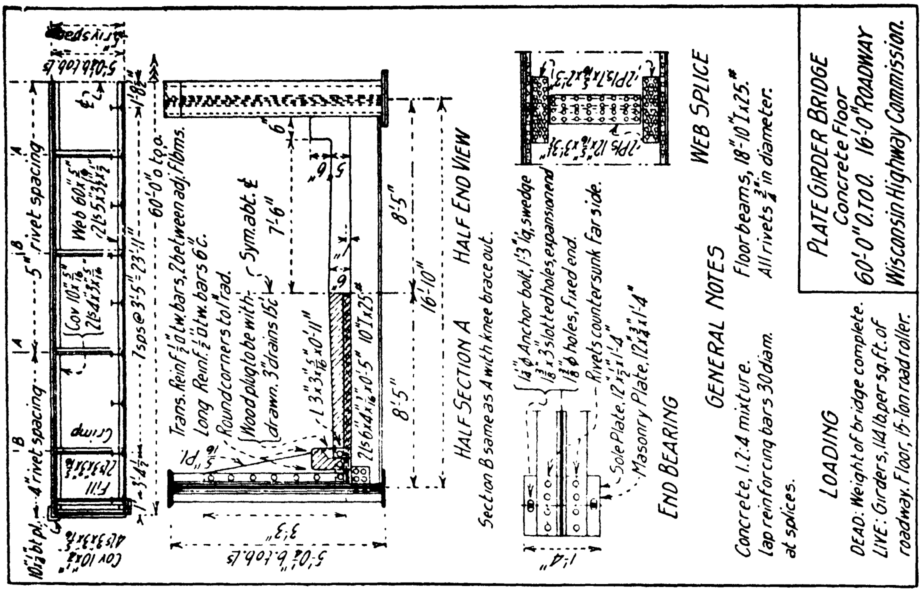
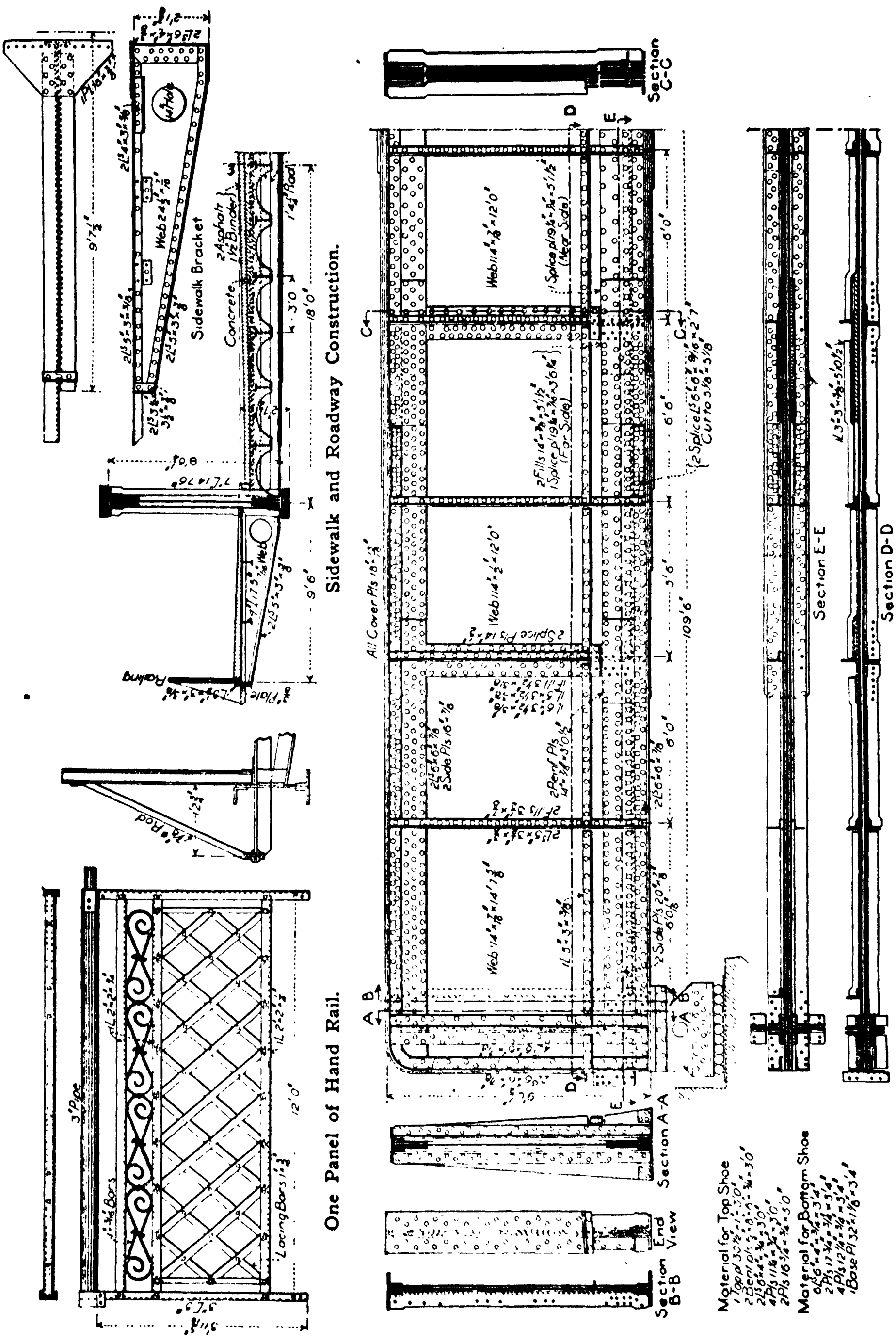


FIG. 15. PLATE GIRDER BRIDGE.



Elevation of Girder Showing Flange Splices.

FIG. 16. DETAILS OF 109-FT. PLATE GIRDER HIGHWAY BRIDGE. (Engineering Record, May 21, 1910.)

PLATE GIRDERS.—Plate girders are frequently used for highway bridges. Where the conditions will permit deck plate girder bridges are to be preferred to through plate girder bridges for highway service. The details of plate girders when used for highway bridges are essentially the same as when used for railway bridges, which see.

Details of a steel through plate girder highway bridge as designed by the Wisconsin Highway Commission are shown in Fig. 15. Standard plans have been prepared for spans from 35 ft. to 80 ft., varying by 5-ft. intervals, and for 16-ft., 18-ft. and 20-ft. roadway. Spans of 35 ft. to 60 ft. inclusive have webs 60 in. by $\frac{5}{16}$ in.; the 65-ft. and 70-ft. spans have webs 66 in. by $\frac{5}{16}$ in.; the 75-ft. spans have a web 66 in. to 72 in. by $\frac{3}{8}$ in., while the 80-ft. spans have a web 72 in. to 78 in. by $\frac{3}{8}$ in. For weights of plate girder bridges, see first part of this chapter.

Details of a 109-ft. span through-plate girder highway bridge built over the D. L. & W. R. R. tracks in Jersey City, N. J., are given in Fig. 16. The girders were designed for a live load of 100 lb. per sq. ft. on roadway and sidewalk; while the roadway floor was designed for a live load of 100 lb. per sq. ft. and two 12,000 lb. axle loads spaced 10 ft. apart with an allowance of 25 per cent for impact. The expansion end is carried on 4-in. rollers. The concrete has a minimum thickness of 4 in. and is covered with $1\frac{1}{2}$ in. of binder and 2 in. of asphalt. Each main girder weighed 112,000 lb.; and the total weight of steel in the bridge was about 403,000 lb.

LOW RIVETED TRUSS BRIDGES.—Low riveted bridges are made with either Warren or Pratt trusses, the Warren truss usually being preferred. The upper chords should be made of two angles and a plate, two channels laced, or two channels with a top cover plate and lacing on the bottom side of the member. The lower chord and the web members are made of two angles placed in the same relative positions as in the upper chords.

Details of a low riveted truss bridge with a reinforced concrete floor carried on steel stringers or joists, as designed by the Iowa Highway Commission are shown in Fig. 17. The commission has prepared standard plans for spans from 35 ft. to 85 ft. and with 16-ft. and 18-ft. roadway. Spans over 65 ft. in length have one end supported on rockers. Spans 65 ft. or less in length have one end supported on sliding plates.

Details of a low riveted truss bridge with a reinforced concrete floor carried directly on the floorbeams, as designed by the Iowa Highway Commission, are shown in Fig. 18. The commission has prepared standard plans for spans from 35 ft. to 100 ft. and with 16-ft. and 18-ft. roadway. Spans more than 65 ft. in length have one end supported on rockers. Spans 65 ft. or less in length have one end supported on sliding plates. The reinforced concrete floor slabs have a thickness of $7\frac{1}{2}$ in. for an 8-ft. span, of 8 in. for a 9-ft. span, and of $8\frac{1}{2}$ in. for a 10-ft. span. The slabs are reinforced top and bottom with $\frac{3}{4}$ in. square bars spaced 9 in. centers and $1\frac{1}{4}$ in. from face of slab. Transverse bars $\frac{1}{2}$ in. sq. are spaced about 2 ft. centers with one bar over the floorbeam.

Details of a low riveted truss bridge with a reinforced concrete floor as designed by the Michigan Highway Commission are given in Fig. 19. The Commission has prepared standard plans for spans from 50 ft. to 100 ft. by 5-ft. intervals.

The riveted low truss highway bridge with an inclined upper chord shown in Fig. 20 is built by the American Bridge Company for locations requiring an artistic and serviceable bridge at a moderate cost. This bridge has been built with six panels and with spans of 90, 96 and 102 ft. The bridge in Fig. 20 has a 20-ft. roadway and was designed for a dead load of 930 lb. per lineal foot of bridge, and a live load of 2,400 lb. per lineal foot of bridge. The total weight of the steel in this bridge, exclusive of joists and fence is, approximately, 57,000 lb. The floorbeams are rolled I-beams and are riveted below the chords. The top chords are made of two channels with a top cover plate, the lower edges of the channels being fastened together with tie plates—lacing is much better practice. The bottom chord is composed of two angles, with tie plates—tie plates are all right for this member. The web members are made of 2 or 4 angles laced, as shown. Rods, not shown, are used for the lower lateral system.

Details of a low riveted truss bridge with a reinforced concrete floor as designed by the Wisconsin Highway Commission are given in Fig. 21. Standard plans have been prepared for spans from 35 ft. to 85 ft., and with 16-ft. and 18-ft. roadway. One end of all spans is carried on sliding plates as shown.

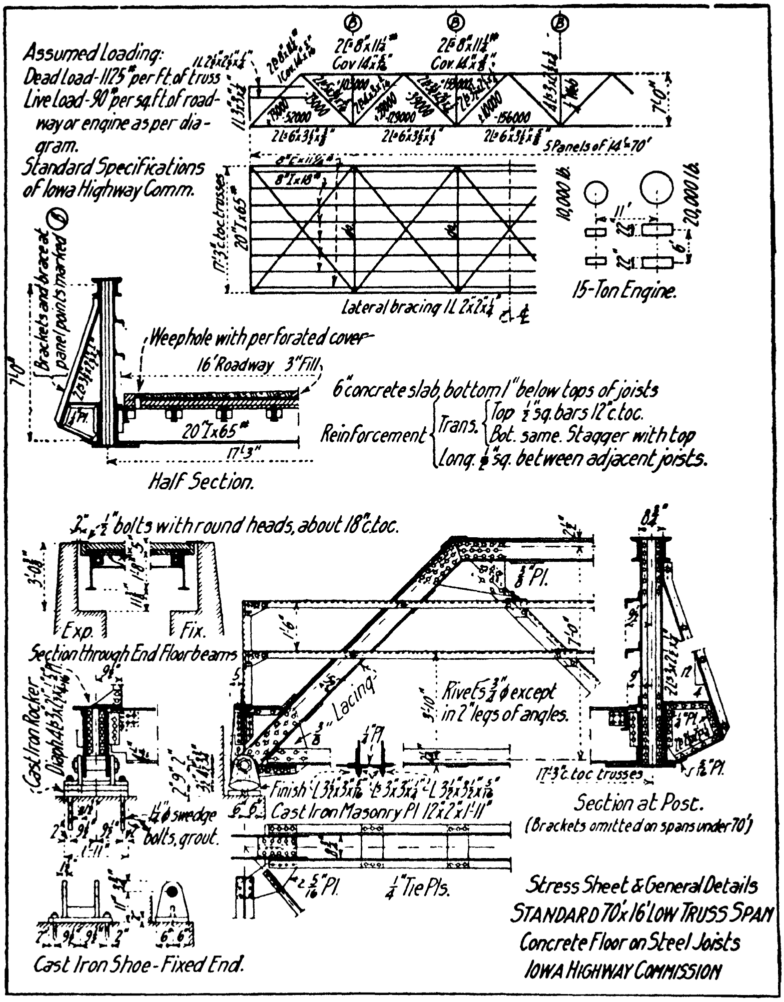


FIG. 17. LOW TRUSS SPAN WITH STRINGERS.

Depth and Panel Length of Low Trusses.—The depths and number of panels in Iowa Highway Commission low truss bridges with joists are as follows: 35 ft. and 40 ft. span, 3 panels, 6 ft. deep; 45 ft. and 50 ft. spans, 3 panels, 6½ ft. deep; 60 ft. and 65 ft. span, 4 panels, 7 ft. deep; 70 ft. span, 5 panels, 7 ft. deep; 80 ft. and 85 ft. span, 5 panels, 8 ft. deep. For low truss bridges without joists, 35 ft. span, 4 panels, 6 ft. deep; 40 ft. span, 5 panels, 6 ft. deep; 45 ft. span, 5 panels, 6½ ft. deep; 50 ft. and 55 ft. span, 6 panels, 6½ ft. deep; 60 ft. span, 7 panels, 7 ft. deep; 65 ft. and 70 ft. span, 8 panels, 7 ft. deep; 75 ft. span, 9 panels, 7½ ft. deep; 80 ft. span, 10 panels, 8 ft. deep; 85 ft. span, 10 panels, 8½ ft. deep; 90 ft. span, 10 panels, 9 ft. deep; 95 ft. span, 10 panels, 9½ ft. deep; 100 ft. span, 10 panels, 10 ft. deep.

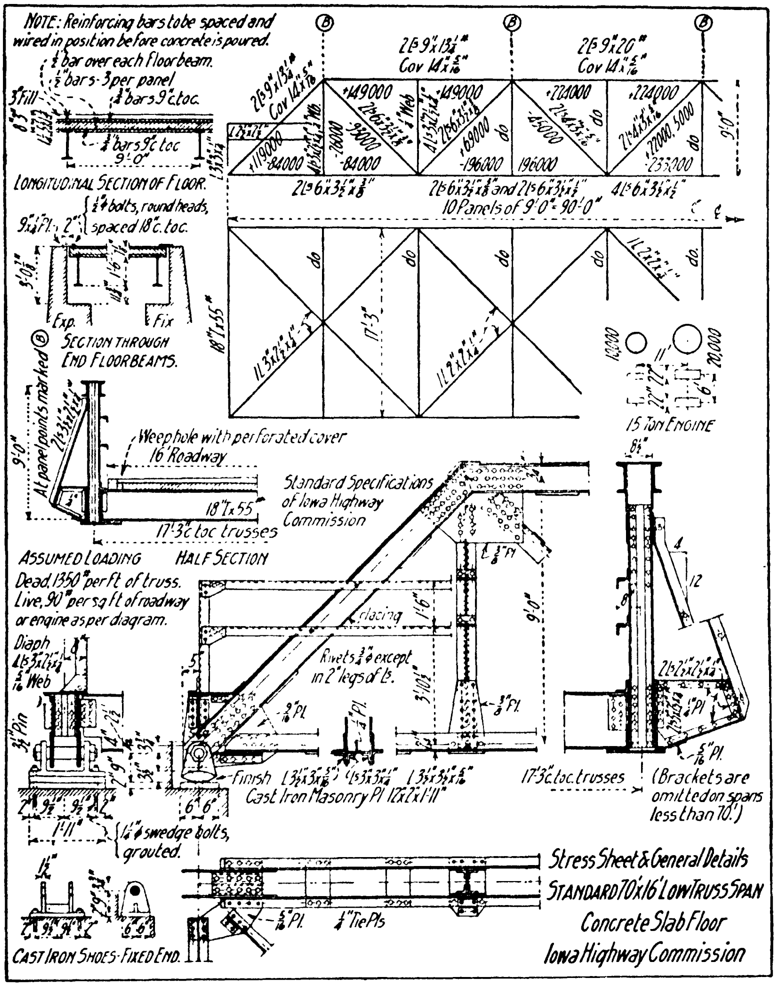


FIG. 18. LOW TRUSS SPAN WITHOUT STRINGERS.

The depths and number of panels in Wisconsin Highway Commission low truss bridges with joists are as follows: 35 ft. span, 3 panels, 4½ ft. deep; 40 ft. span, 3 panels, 5 ft. deep; 45 ft. span, 3 panels, 5½ ft. deep; 50 ft. span, 4 panels, 5½ ft. deep; 55 ft. span, 4 panels, 6 ft. deep; 60 ft. span, 4 panels, 6½ ft. deep; 65 ft. span, 5 panels, 7 ft. deep; 70 ft. span, 5 panels, 7½ ft. deep; 75 ft. span, 5 panels, 8 ft. deep; 80 ft. span, 5 panels, 8½ ft. deep; 85 ft. span, 6 panels, 9 ft. deep.

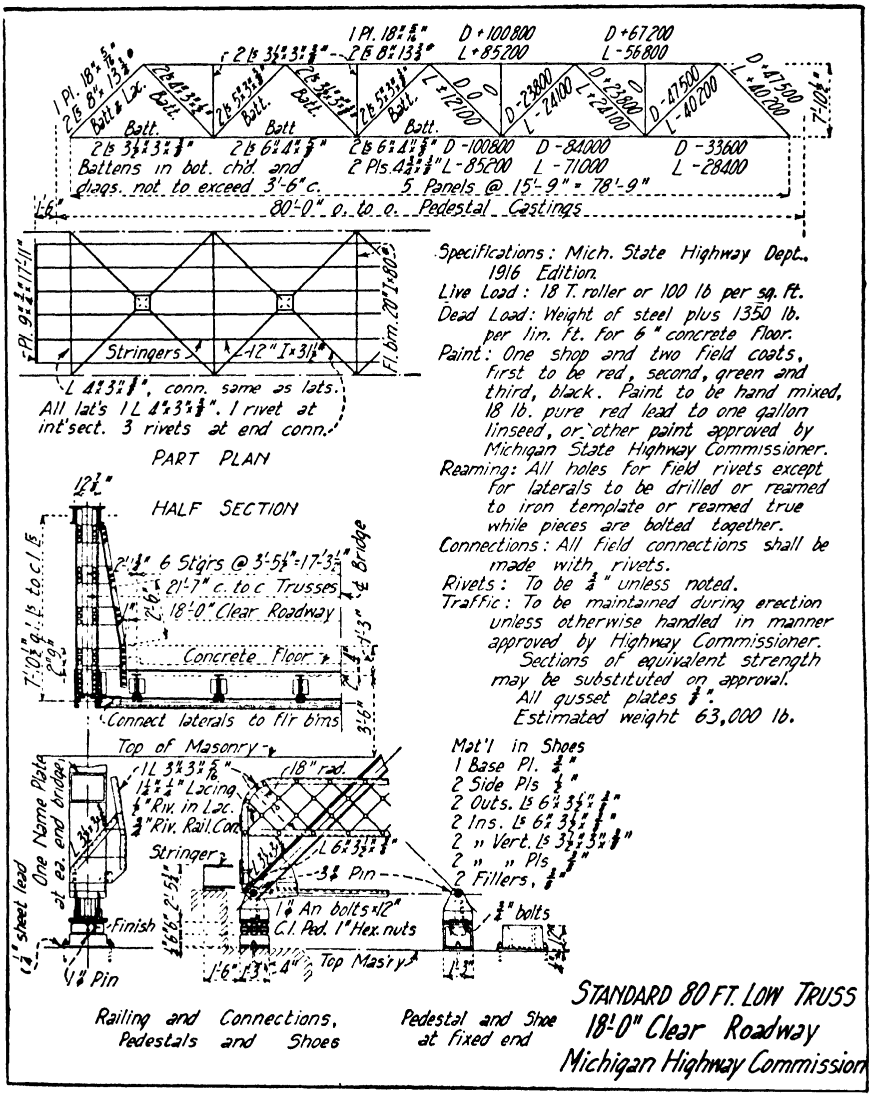


FIG. 19. LOW TRUSS SPAN WITH STRINGERS.

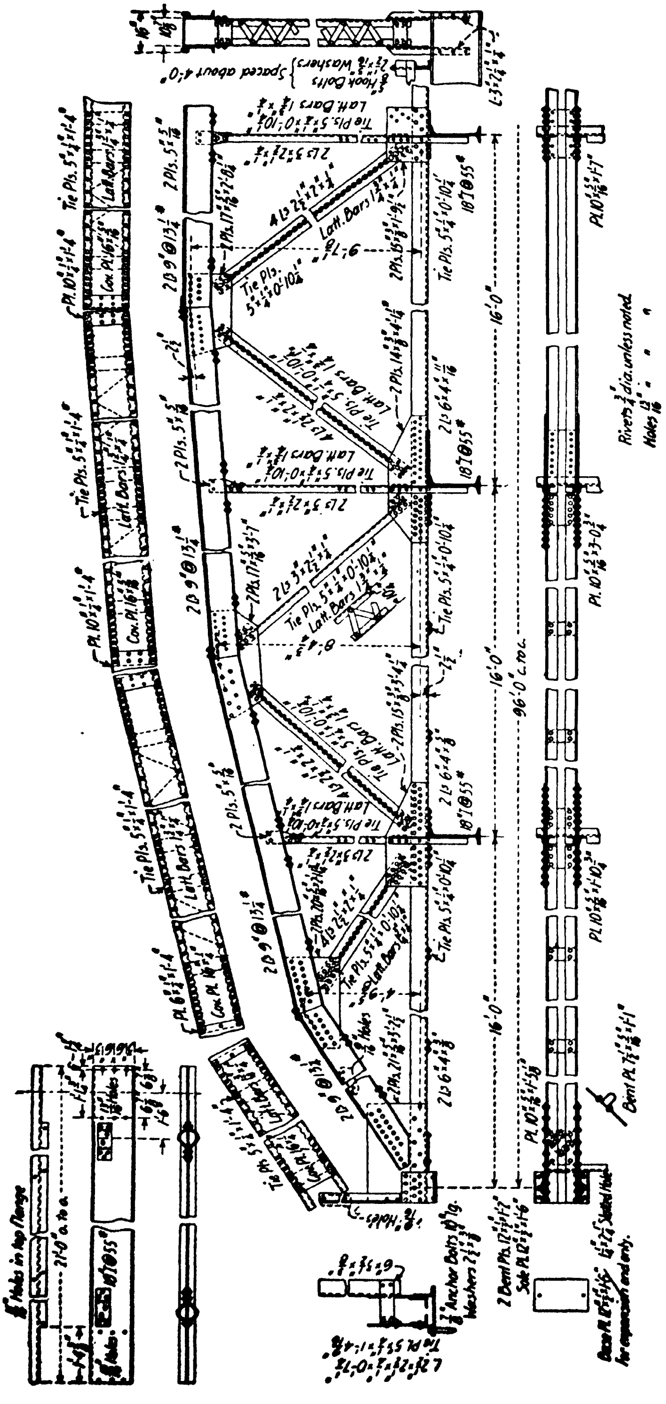


FIG. 20. DETAILS OF LOW TRUSS RIVETED HIGHWAY BRIDGE WITH INCLINED CHORDS. (AMERICAN BRIDGE COMPANY.)

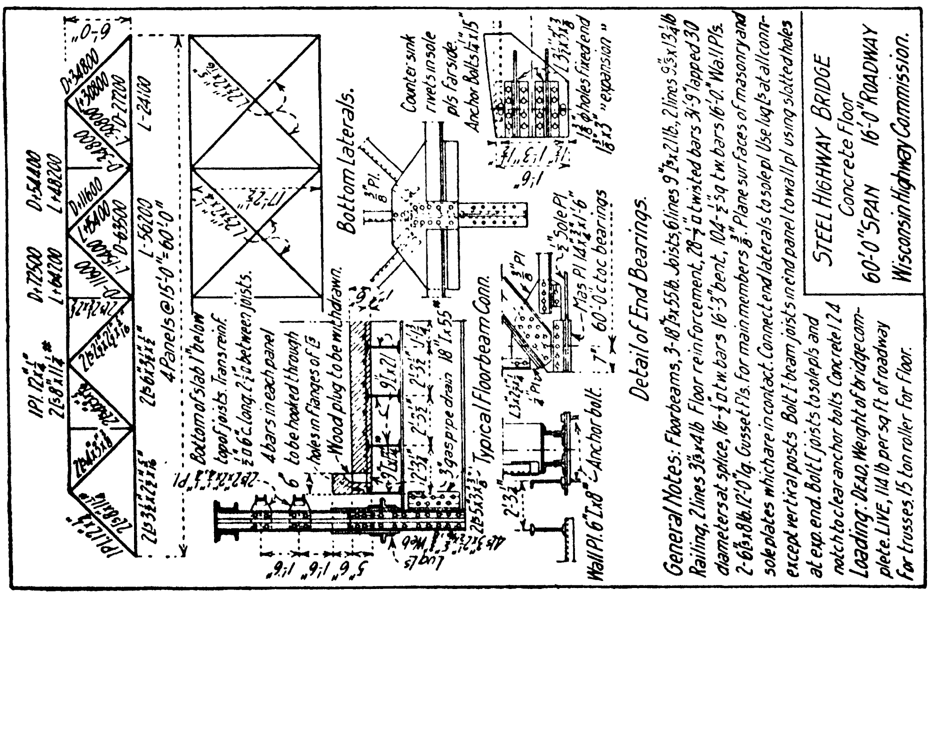


FIG. 21. LOW TRUSS STEEL HIGHWAY BRIDGE.

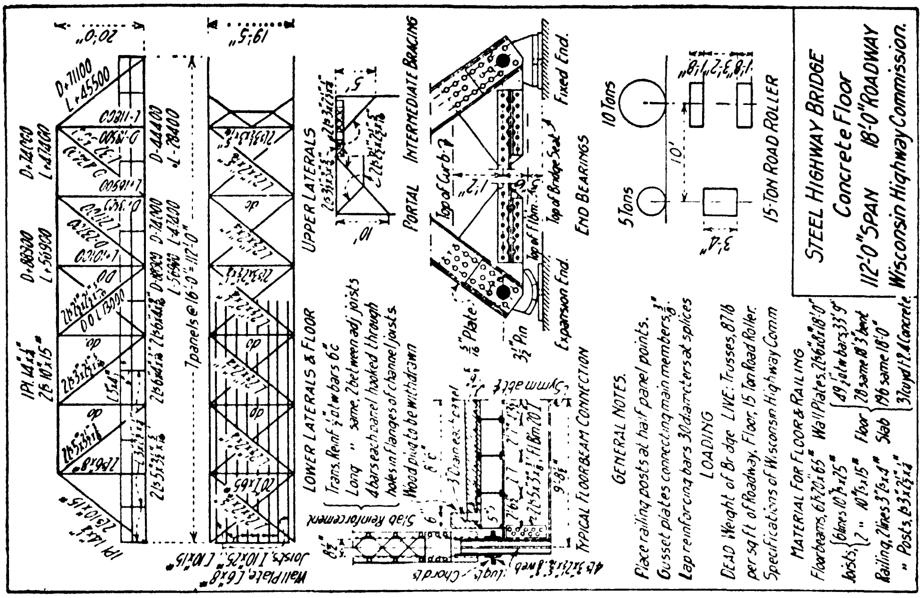


FIG. 22. HIGH TRUSS STEEL HIGHWAY BRIDGE.

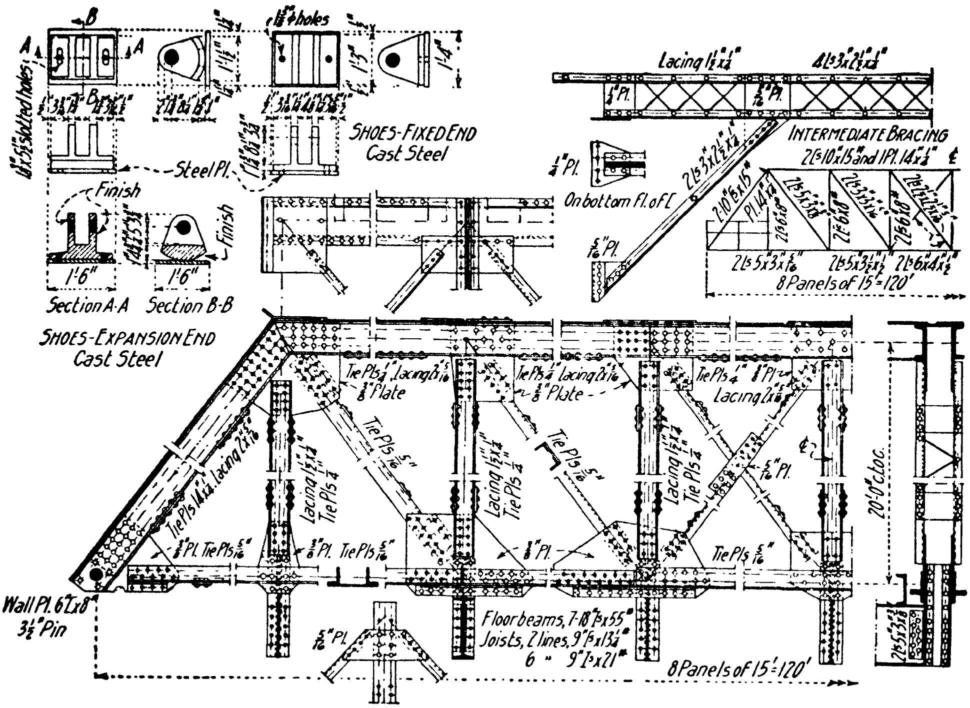


FIG. 23. DETAIL PLANS OF THROUGH HIGH TRUSS SPAN. WISCONSIN HIGHWAY COMMISSION.

HIGH TRUSS STEEL HIGHWAY BRIDGES.—Through truss bridges with spans of from 80 to 170 ft., are built with parallel chords and preferably with riveted joints. For spans of from 160 to 220 ft. bridges are usually built of the Pratt type with inclined upper chord (camel-back) trusses. Above 220 ft., bridges are usually built with the Petit type of truss. The above limits are approximate only. For long span bridges the inclined chord truss with K-bracing is rapidly taking the place of the Petit truss. High truss pin-connected bridges should never be built with less than five panels.

Types of bridge adopted in the American Bridge Company's standards are as follows:

Pratt, pin-connected trusses	80 to 168 ft. span
Pratt, riveted trusses	80 to 168 ft. span
Warren, quadrangular, riveted trusses	80 to 152 ft. span
Inclined chord Pratt (camel-back), pin-connected trusses	168 to 220 ft. span
Petit trusses, pin-connected	220 ft. span and over

Examples of High Truss Highway Bridges.—Details of a high truss steel highway bridge as designed by the Wisconsin Highway Commission are shown in Fig. 22 and Fig. 23. Standard plans have been prepared for spans of 90 ft. to 150 ft., varying by 5-ft. intervals, and a roadway of 16 ft. and 18 ft. All spans have one end carried on rockers as shown. These designs have been worked out very economically by Mr. M. W. Torkelson, bridge engineer, and represent the extreme economy of design that will conform to good practice.

Details of a high truss steel highway bridge as designed by the Iowa Highway Commission are given in Fig. 24. Standard plans have been prepared for spans of 90 ft. to 150 ft. varying by 5-ft. intervals, and a roadway of 16 ft. and 18 ft. All spans have one end carried on rockers as shown. The designs are well worked out with the exception of the collision strut in the first panel, which should be omitted.

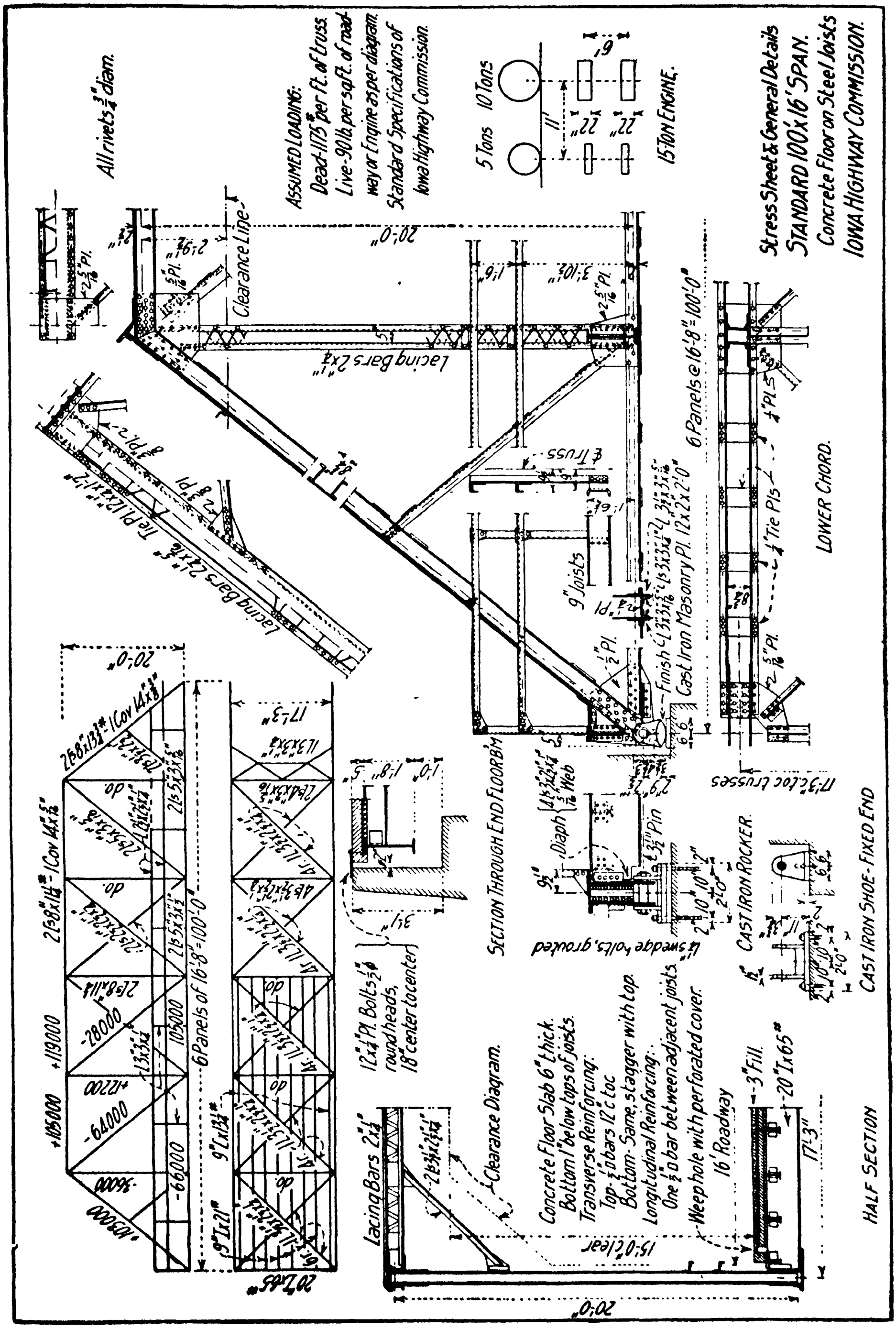


FIG. 24. STANDARD HIGH THROUGH TRUSS SPAN.

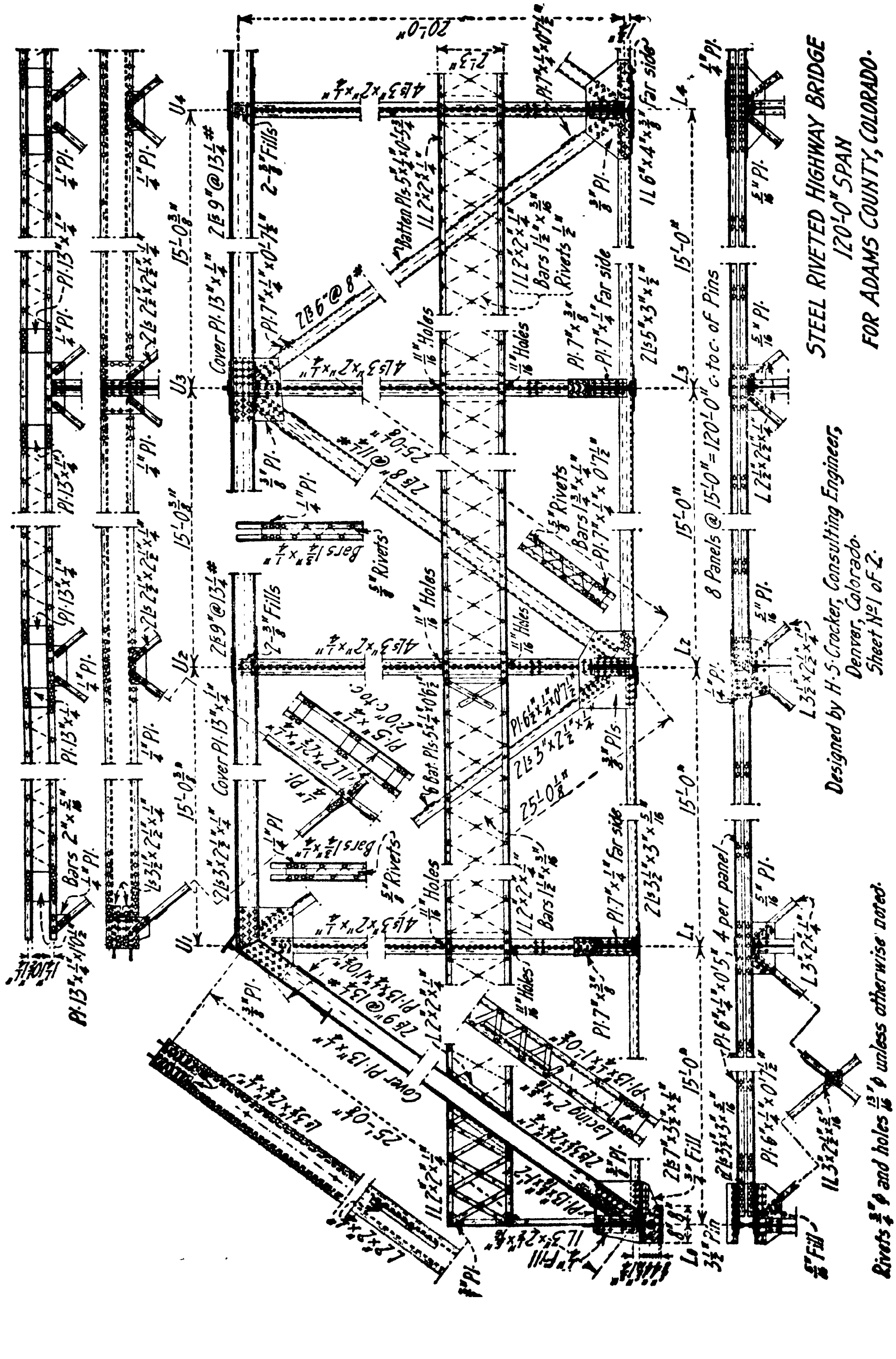


FIG. 25. DETAILS OF A THROUGH RIVETED HIGHWAY BRIDGE.

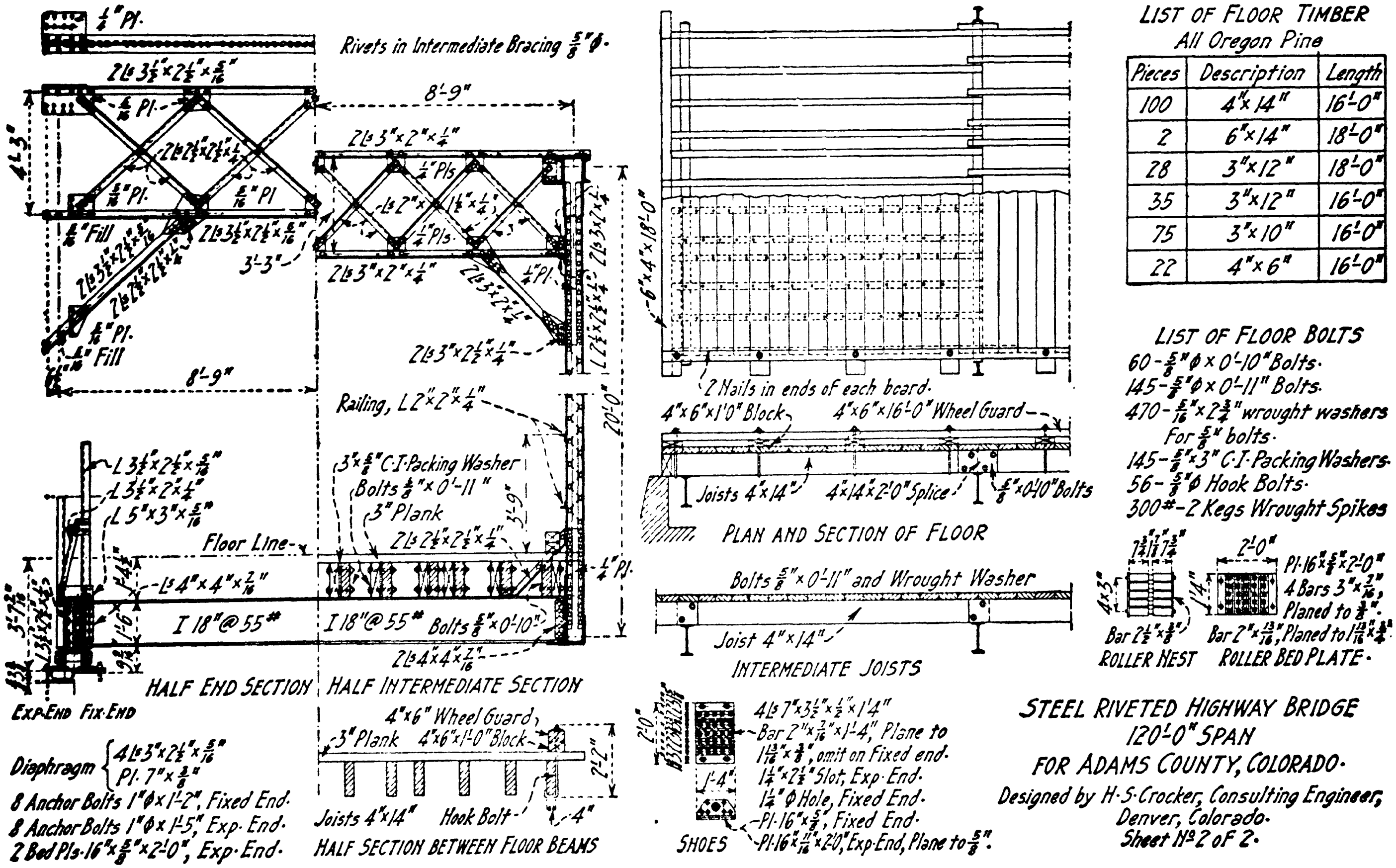


FIG. 26. DETAILS OF A THROUGH RIVETED HIGHWAY BRIDGE.

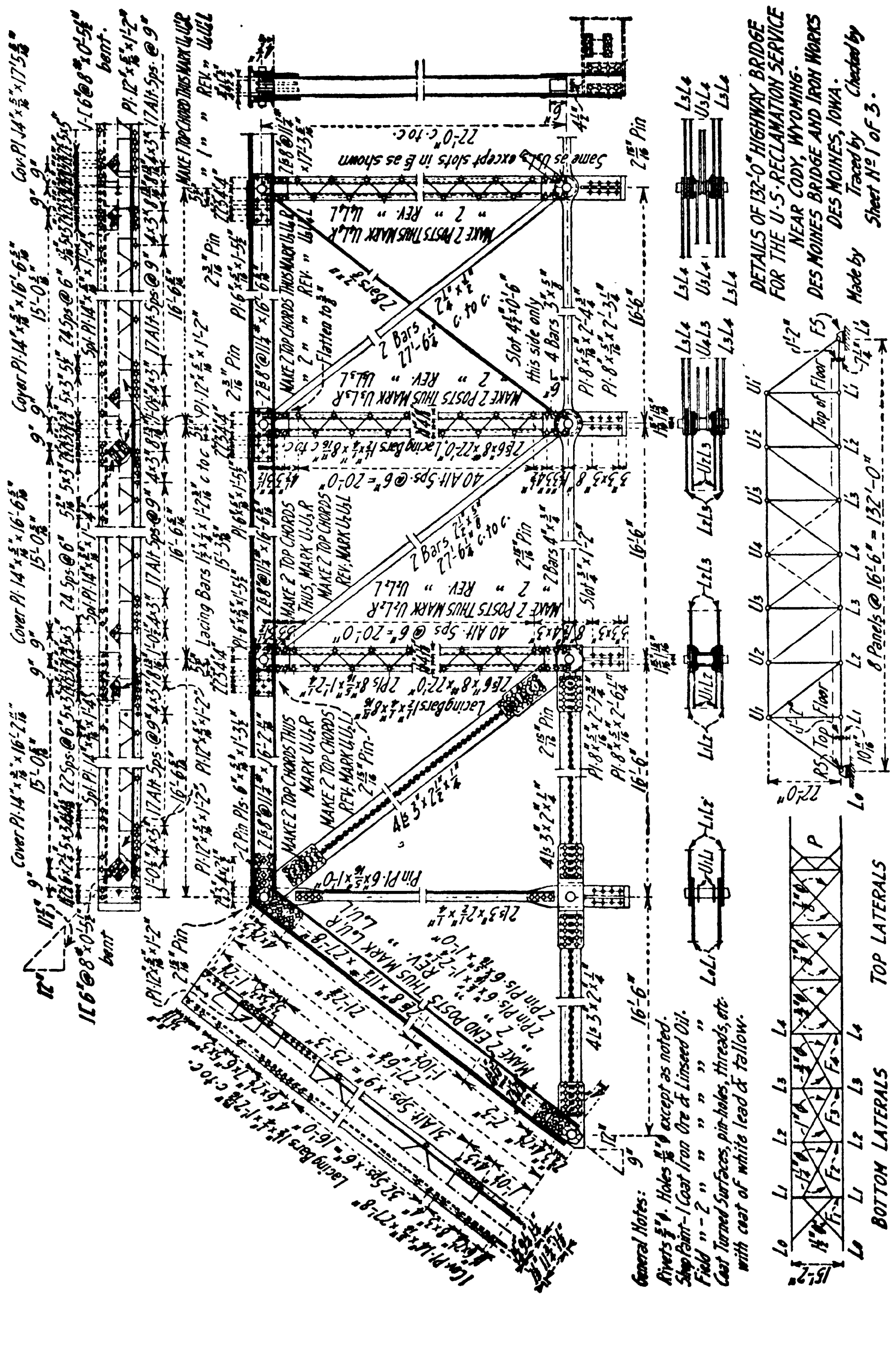


FIG. 27. DETAILS OF A PIN-CONNECTED HIGHWAY BRIDGE.

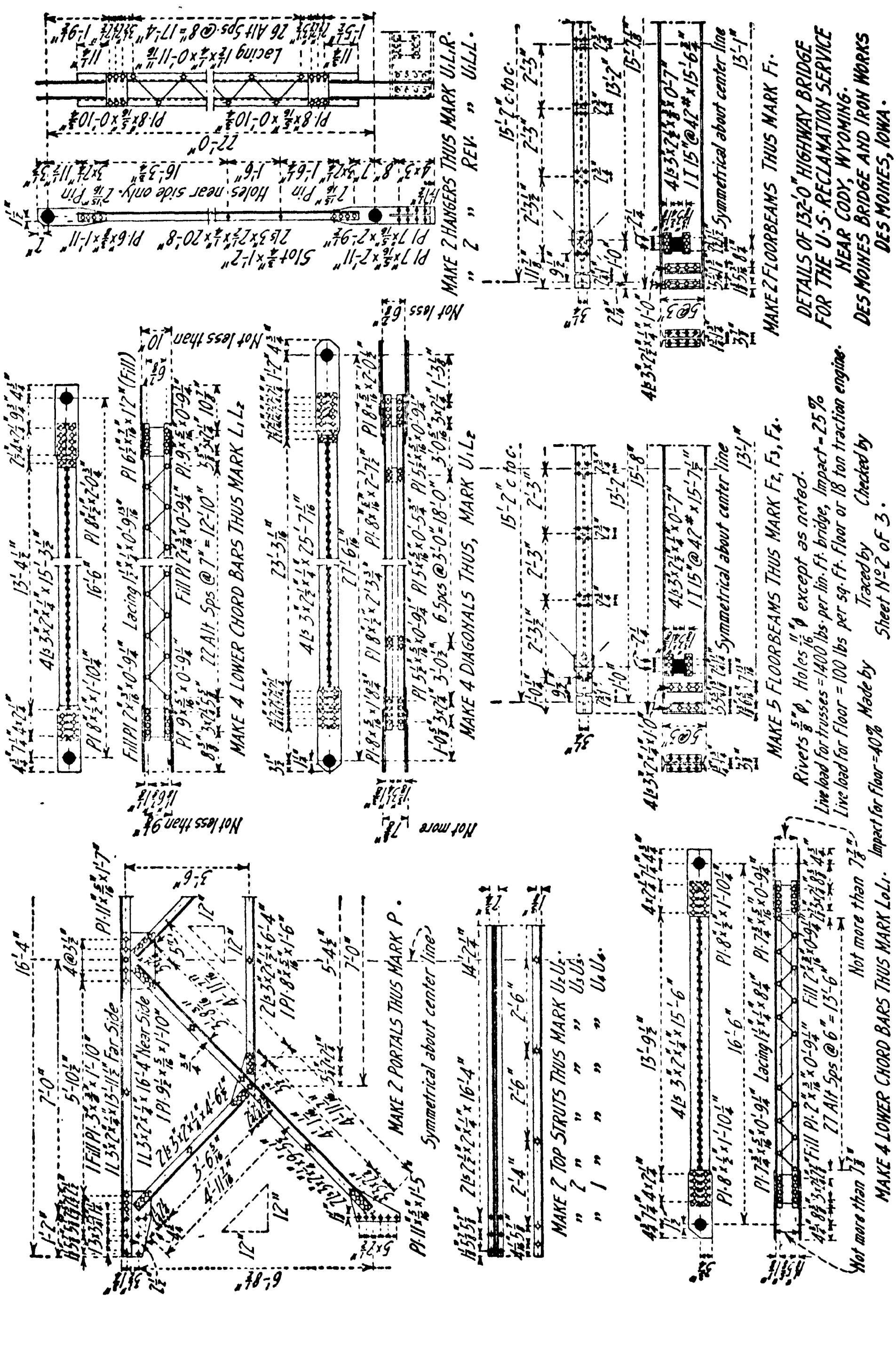


FIG. 28. DETAILS OF A PIN-CONNECTED HIGHWAY BRIDGE.

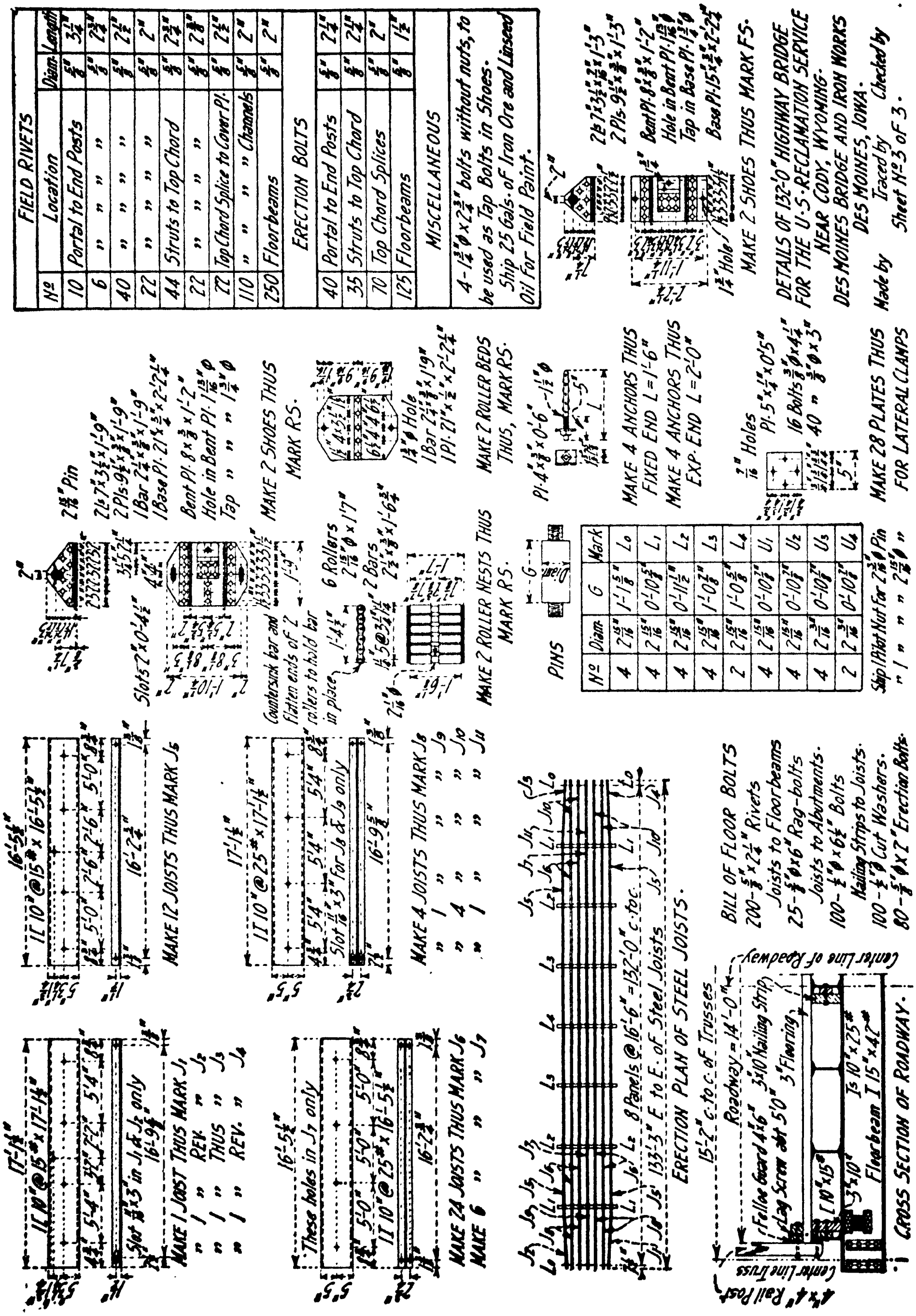


FIG. 29. DETAILS OF A PIN-CONNECTED HIGHWAY BRIDGE.

The details of a riveted truss highway bridge for light country traffic designed by Mr. H. S. Crocker, Consulting Engineer, Denver, Colo., are given in Fig. 25 and Fig. 26. The details of a pin-connected truss highway bridge designed for country traffic are given in Fig. 27, Fig. 28 and Fig. 29. Both of these bridges represent standard practice in the design of steel highway bridges for light country traffic. For additional examples of steel highway bridges, see the author's "The Design of Highway Bridges."

Economic Depth and Panel Length of Trusses.—The economic depth and panel length of trusses is not capable of mathematical calculation. The minimum depth is determined by the required clear head room, which varies from 12½ to 15 ft. Short panel lengths give heavy trusses and light floor systems; while long panels give light trusses and heavy floor systems. For ordinary conditions it is not economical to use panel lengths less than 15 ft. for short spans nor more than 25 ft. for long spans. The minimum depth for through spans is about 16 feet where the floorbeams are placed below the lower chords. To make a stiff structure, the depth should be sufficient to permit the placing of the floorbeams above the lower chords and to permit of efficient portal and sway bracing. Experience has shown that the most economical conditions occur when the angle θ, the tangent of which is the panel length divided by the depth, is about 40 degrees. The top chord points of bridges with inclined chords should be approximately on a parabola passing through the pin at the hip.

Depth and Panel Length of High Trusses.—The depths and number of panels in Iowa Highway Commission high truss riveted bridges are as follows: Pratt, riveted trusses, 90-ft. span, 5 panels, 20 ft. deep; 100-ft. and 110-ft. spans, 6 panels, 20 ft. deep; 120-ft. span, 7 panels, 20 ft. deep; 140-ft. span, 8 panels, 21 ft. deep. The depths and number of panels in Wisconsin Highway Commission high truss riveted bridges are as follows: 90-ft. and 96-ft. span, 6 panels, 18 ft. deep; 100-ft. span, 6 panels, 20 ft. deep; 105-ft. span, 7 panels, 20 ft. deep; 120-ft. span, 8 panels, 20 ft. deep; 128-ft. span, 8 panels, 21 ft. deep; 140-ft. span, 8 panels, 20 ft. deep at hip and 27 ft. deep at center; 150-ft. span, 8 panels, 20 ft. deep at hip and 28 ft. deep at center.

The depths and number of panels in American Bridge Company's high truss bridges are as follows: Riveted and pin-connected trusses with parallel chords, 80-ft. to 90-ft. span, 5 panels, depth equal to panel length; 90- to 120-ft. span, 6 panels, depth equal to panel length; 120-ft. span to 140-ft. span, 7 panels, depth equal to panel length, 120-ft. to 168-ft. span, 8 panels, ratio of depth to panel length 1.1. For bridges with inclined chords with spans of 162 ft. to 180 ft., 9 panels, and ratios of depth to panel length of 1.0, 1.16, 1.25 and 1.29; 190-ft. to 220-ft. span, 9 panels, and ratios of depth to panel length of 1.0, 1.24, 1.28 and 1.43. For Petit trusses, 240-ft. to 276-ft. span, 12 panels, and ratios of depths to panel length of 1.0, 1.4, 1.6 and 1.7; 294-ft. to 322-ft. span, 14 panels, and ratios of depth to panel length of 1.0, 1.36, 1.60, 1.8 and 2.0.

SHOES AND PEDESTALS.—The bridge rests on shoes or pedestals, the loads being transferred to the shoes in pin-connected bridges by means of pins, and through the riveted joints in riveted bridges. The shoes at the expansion ends of the bridge are placed on smooth sliding plates for bridges of less than, say, 65-ft. span, and on nests of rollers or rockers for spans of greater length. The action of the rollers under the expansion ends of riveted bridges will be much more satisfactory if the shoes are pin-connected to the truss the same as for pin-connected trusses. Rollers should be made with as large diameters as practicable in order to reduce the pressure on the base plate and also to reduce the resistance to movement. Experience shows that even for light bridges rollers smaller than 3 in. diameter are practically worthless. To economize space, segmental rollers, as shown in Fig. 35, Chapter IV, are often used for heavy spans.

It is usual to specify that a movement produced by a variation of 150 degrees Fahr. be provided for. The coefficient of expansion of steel is approximately 0.0000067 per degree Fahr., which makes it necessary to provide for approximately one inch of movement for each 80 ft. of bridge span.

Where both bridge seats are of the same height, the fixed end is carried on cast iron pedestal blocks. The blocks are usually made with recesses (honeycombed) to reduce the weight.

The Illinois, Iowa and Wisconsin Highway Commissions use rockers in the place of rollers for highway bridges. Details of rockers are shown in Fig. 17, Fig. 18, Fig. 23, and Fig. 24. The specifications of the Illinois Highway Commission contain the provision that rockers shall be made of cast iron as specified. They shall have a thickness of not less than 2½ in. for spans of 45 ft. or less, and a thickness of 3 in. for spans exceeding 45 ft. in length, but in no case shall the unit compressive stress exceed 9,000-40 l/r lb. per sq. in. All rockers shall have bearing surfaces turned to a uniform radius and smooth surface and shall be provided with two 2-in. holes through the web to facilitate handling.

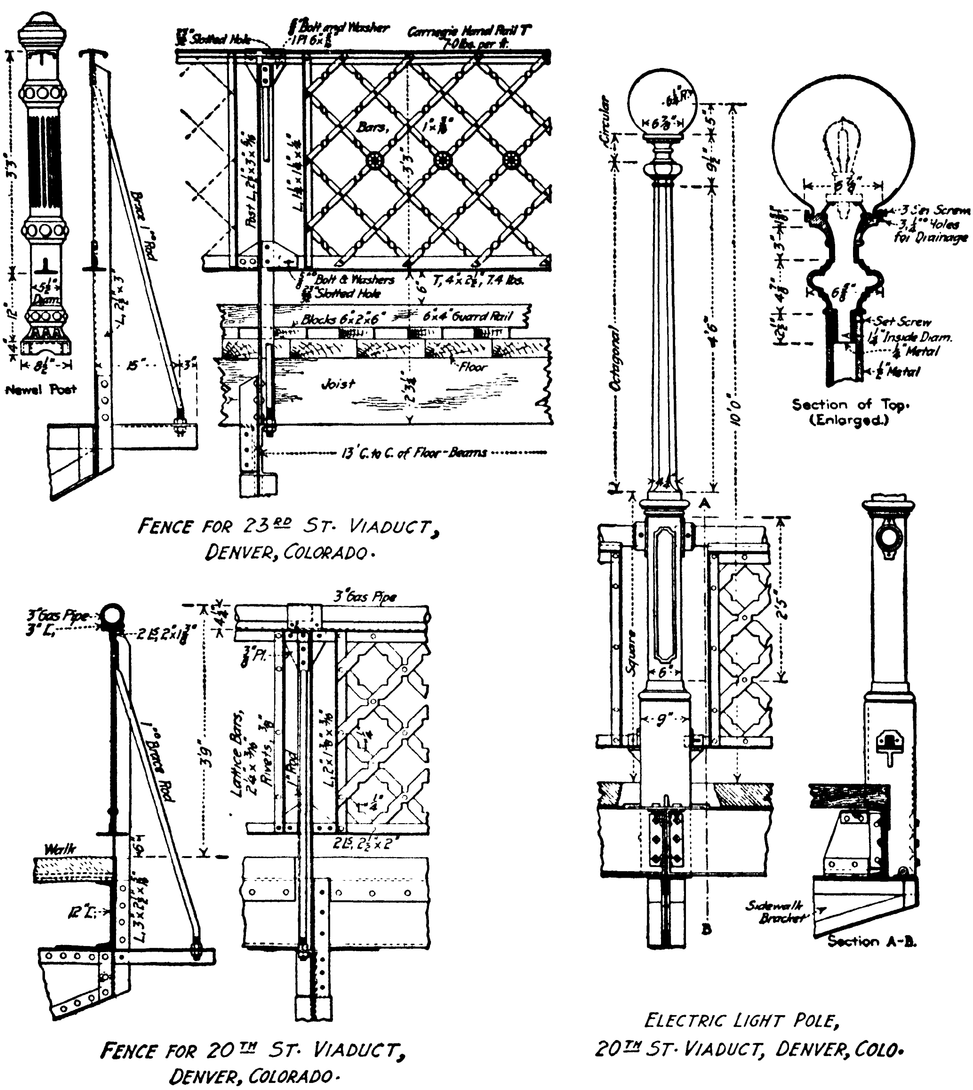


FIG. 30. STEEL FENCE FOR HIGHWAY BRIDGES.

FENCE AND HUB GUARDS.—The fence on steel bridges is commonly made of two lines of channels or two lines of angles with angle posts. Posts should not be spaced farther apart than 8 ft. to 10 ft.

A gas pipe railing with gas pipe posts is in frequent use. The posts should be spaced not more than 8 ft. apart. Details of the fence and light poles for the 20th St. Viaduct, and the fence on 23d St. Viaduct, Denver, Colo., designed by Mr. H. S. Crocker, consulting engineer, are shown in Fig. 30.

GENERAL SPECIFICATIONS FOR STEEL HIGHWAY BRIDGES.*

BY

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M. Am. Soc. C. E.

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PART I. DESIGN.

GENERAL DESCRIPTION.

1. **Classes.**—Bridges under these specifications are divided into eight classes, as follows:
Class A.—For city traffic.
Class B.—For suburban or interurban traffic with heavy electric cars.
Class C.—For country roads with ordinary traffic and light electric cars.
Class D_1.—For country roads with heavy traffic.
Class D_2.—For country roads with light traffic.
Class E_1.—For heavy electric street railways only.
Class E_2.—For medium electric street railways only.
Class E_3.—For light electric street railways only.

2. **Material.**—All parts of the structure shall be of rolled steel, except the flooring, floor joists and wheel guards, when wooden floors are used. Cast iron or cast steel may be used in the machinery of movable bridges, for wheel guards, and for bed plates and rockers.

3. **Types of Truss.**—The following types of bridges are recommended:
Spans up to 30 ft.—Rolled beams.
Spans from 30 to 80 ft.—Riveted plate girders, or riveted low trusses for classes A, B, E_1, E_2 and E_3; and riveted low trusses for classes C, D_1 and D_2.
Spans 80 to 160 ft.—Riveted or pin-connected high trusses.
Spans 160 to 200 ft.—Pin-connected trusses of the Pratt type with inclined chords.
Spans over 200 ft.—Pin-connected trusses of the Petit type or K-type.

4. **Length of Span.**—In calculating the stresses the length of span shall be taken as the distance between centers of end pins for pin-connected trusses, centers of end bearing plates for riveted trusses and for girders, and center to center of trusses for floorbeams.

5. **Form of Trusses.**—The form of truss shall preferably be as given in paragraph 3. In through trusses the end vertical suspenders and the two panels of the lower chord at each end shall be made rigid members if the wind load produces a reversal of stress in the lower chord. In through bridges the floorbeams shall be riveted above or below the lower chord pins.

6. **Lateral Bracing.**—All lateral and sway bracing shall preferably, and all portal bracing must be, made of shapes capable of resisting compression as well as tension, and shall have riveted connections. Low trusses and through plate girders shall be stayed by knee braces or gusset plates at each floorbeam.

7. **Spacing of Trusses.**—For bridges carrying electric cars the clear width from the center of the track shall not be less than 7 ft. at a height exceeding one foot above the track where the tracks are straight, and an equivalent distance when the tracks are curved. The distance between centers of trusses shall in no case be less than one-twentieth of the span between the centers of end-pins or shoes, and shall preferably not be less than one-twelfth of the span.

8. **Head Room.**—For classes A, B, C, D_1, E_1, E_2 and E_3 the clear head room for a width of eight (8) ft. on each track, or eight (8) ft. on the center line of the bridge shall not be less than 15 ft., and for class D_2 not less than 12½ ft.

9. **Footwalks.**—Where footwalks are required, they shall generally be placed outside of the trusses and be supported on longitudinal beams resting on overhanging steel brackets.

10. **Handrailing.**—A strong and suitable handrailing shall be placed at each side of the bridge and be rigidly attached to the superstructure.

11. **Trestle Towers.**—Trestle bents shall preferably be composed of two supporting columns, two bents forming a tower; each tower thus formed shall be thoroughly braced in both directions and have struts between the feet of the columns. The feet of the columns must be secured to an anchorage capable of resisting one and one-half times the specified wind forces (§89).

* Reprinted from the author's "The Design of Highway Bridges of Steel, Timber and Concrete," Second Edition, 1920.

Each tower shall have a sufficient base, longitudinally to be stable when standing alone, without other support than its anchorage. Tower spans for high trestles shall not be less than 30 ft.

12. **Proposals.**—Contractors in submitting proposals shall furnish complete stress sheets, general plans of the proposed structures, and such detail drawings as will clearly show the dimensions of all the parts, modes of construction and sectional areas.

13. **Drawings.**—Upon the acceptance and the execution of the contract, all working drawings required by the engineer shall be furnished free of cost (§168).

14. **Approval of Plans.**—No work shall be commenced or materials ordered until the working drawings have been approved by the engineer in writing.

FLOOR SYSTEM.

15. **Floorbeams.**—All floorbeams shall be rolled or riveted steel girders, rigidly connected to the trusses at the panel points, or may be placed on the top of deck bridges at panel points. Floorbeams shall preferably be square to the trusses or girders.

16. **Joists and Stringers.**—All joists and stringers of bridges of classes A, B, E_1, E_2 and E_3 shall be of steel. Joists for classes C, D_1 and D_2 may be either of wood or steel as specified. Steel joists shall be securely fastened to the cross floorbeams, and steel stringers shall preferably be riveted to the webs of floorbeams by means of connection angles at least $\frac{7}{16}$ in. thick.

17. **End Spacers for Stringers.**—Where end floorbeams cannot be used, stringers resting on masonry shall have cross-frames at their ends. These frames shall be riveted to girder or truss shoe where practicable.

18. **Wooden Joists.**—Wooden floor joists shall be spaced not more than $2\frac{1}{2}$ ft. centers, and shall lap by each other so as to have a full bearing on the floorbeams, and shall be separated $\frac{1}{2}$ in. for free circulation of air. Their width shall not be less than 3 in., or one-fourth the depth in width. The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists in feet divided by four feet. No impact shall be considered in the design of wooden joists, planks or ties. Oak, longleaf yellow pine and Oregon fir shall be designed for a safe bending of 1,500 lb. per sq. in., bearing across the fiber of 400 lb. per sq. in., and shearing along the grain of 140 lb. per sq. in. Outside joists shall be designed for the same live loads as the intermediate joists.

19. **Steel Joists.**—Steel I-beams when used as joists shall have a depth of not less than one-thirtieth of the span, and one-twentieth of the span when used as track stringers. The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists in feet divided by four feet when timber flooring is used, and divided by six feet when a reinforced concrete or other rigid floor is used. Outside joists shall be designed for the same live loads as the intermediate joists.

20. **Floor Plank.**—For single thickness the roadway planks shall not be less than 3 in. thick nor less than one-eighth of the distance between centers of joists, and shall be laid transversely with $\frac{1}{2}$ in. openings and securely spiked to each joist. All plank shall be laid with heart side down. When an additional wearing surface is required it shall be $1\frac{1}{2}$ in. thick, and the lower planks of a minimum thickness of 3 in. shall be laid diagonally with $\frac{1}{2}$ in. openings.

21. Footwalk plank shall be not less than 2 in. thick nor more than 6 in. wide, spaced with $\frac{1}{4}$ in. openings.

All plank shall be laid with heart side down, shall have full and even bearing on and be firmly attached to the joists.

22. **Wheel Guards.**—Wheel guards of a cross-section of not less than 6 in. by 4 in. shall be provided on each side of the roadway. They shall be spliced with half-and-half joints with 6 in. lap, and shall be bolted to the stringers or joist with $\frac{5}{8}$ in. bolts, spaced not to exceed 5 ft. apart.

23. **Solid Floor.**—For bridges of classes A and B a solid floor, consisting of wooden blocks, brick, stone, asphalt, etc., on a concrete bed is recommended. For this case the floor shall consist of buckle plates or corrugated sections or reinforced concrete slabs, and a waterproof concrete (bitumen or cement) bed not less than 3 in. thick for the roadway and 2 in. thick for the footwalk, over the highest point to be covered, not counting rivet or bolt heads. The floor shall be laid with a slope of at least one inch in 10 ft.

Reinforced Concrete Floor.—For the design of reinforced concrete floors, see page 152, and for the distribution of loads on slabs see page 150.

24. Buckle plates shall not be less than $\frac{5}{16}$ in. thick for the roadway and $\frac{1}{4}$ in. thick for the footwalk. The crown of the plates shall not be less than 2 in.

25. For solid floor the curb holding the paving and acting as a wheel guard on each side of the roadway shall be of stone or steel projecting about 6 in. above the finished paving at the gutter. The curb shall be so arranged that it can be removed and replaced when worn or injured. There shall also be a metal edging strip on each side of the footwalk to protect and hold the paving in place.

26. **Drainage.**—Provision shall be made for drainage clear of all parts of the metal work.

27. **Floor of Classes E_1, E_2, and E_3.**—The floors of classes E_1, E_2, and E_3 shall consist of cross-ties not less than 6 in. by 6 in. for stringers spaced 6½ ft.; and larger for greater spacings, they shall be spaced with openings not exceeding 6 in., shall be notched down ½ in., and secured to the supporting stringers by ¾ in. bolts spaced not over 6 ft. apart. The ties shall extend the full width of the bridge on deck bridges, and every other tie shall extend the full width in through bridges to carry the footwalk. Ties shall be designed for the same allowable unit stresses as wooden joists.

There shall be guard timbers not less than 6 in. by 6 in., or 5 in. by 7 in., on each side of each track, with their inner faces not less than 9 in. from the center of the rail. They shall be notched 1 in. over every tie, and shall be spliced over a tie with a half-and-half joint with 6 in. lap. Each guard timber shall be fastened to every third tie and at each splice with a ¾ in. bolt. All heads or nuts on the upper faces of ties or guards shall be countersunk below the surface of the wood.

PART II. LOADS.

28. **Dead Load.**—The dead load will consist of (1) the weight of the metal, and (2) the weight of the timber in the floor, or of the material other than steel. In determining the dead load the weight of oak or other hard wood shall be taken at 4½ lb. per foot board measure, and the weight of pine or other soft woods at 3½ lb. per foot; the weight of asphalt at 130 lb., of concrete and paving brick at 150 lb., and of granite at 160 lb. per cu. ft.

The rails, fastenings, splices and guard timbers of street railway tracks shall be assumed to weigh not less than 100 lb. per lineal foot of track.

29. **Live Load.**—The bridges of different classes shall be designed to carry, in addition to their own weight and that of the floor, a moving load, either uniform or concentrated, or both, as specified below, placed so as to give the greatest stress in each member.

Class A. For City Traffic.—For the floor and its supports, on any part of the roadway or on each of the street car tracks, a concentrated load of 24 tons on two axles 10 ft. centers and 5 ft. gage (assumed to occupy 12 ft. in width for a single line or 22 ft. for a double line), and upon the remaining portion of the floor, a load of 125 lb. per sq. ft. and a concentrated load as for class D_1. Sidewalks a load of 100 lb. per sq. ft.

Loads for the trusses as per Table I.

Class B. For Suburban or Interurban Traffic.—For the floor and its supports, on any part of the roadway, a concentrated load of 12 tons on two axles 10-ft. centers and 5-ft. gage (assumed to occupy a width of 12 ft.), or on each street car track a concentrated load of 24 tons on two axles 10-ft. centers; and on the remaining portion of the floor, a load of 125 lb. per sq. ft. and a concentrated load as for class D_1. Sidewalks a load of 100 lb. per sq. ft.

Loads for the trusses as per Table I.

Class C. For Highway and Light Interurban Traffic.—For the floor and its supports, on any part of the roadway, a concentrated load of 12 tons on two axles 10-ft. centers and 5-ft. gage (assumed to occupy a width of 12 ft.), or on each street car track a concentrated load of 18 tons on two axles 10-ft. centers; and upon the remaining portion of the floor, a load of 125 lb. per sq. ft. and a concentrated load as for class D_1. Sidewalks a load of 100 lb. per sq. ft.

Loads for the trusses as per Table I.

Class D_1. Heavy Country Bridges.—For the floor and its supports, a load of 125 lb. per sq. ft. of total floor surface or a 20-ton motor truck with axles spaced 12 ft. and wheels with 6 ft. centers, with 14 tons on rear axle and 6 tons on front axle. The truck to occupy a space 10 ft. wide and 32 ft. long. The rear wheels to have a width of 20 in.

Loads for the trusses as per Table I. No bridge, however, to be designed for a load of less than 1,000 lb. per lineal foot of bridge.

Class D_2. Ordinary Country Bridges.—For the floor and its supports, a load of 100 lb. per sq. ft. of total floor surface of a 15-ton motor truck with axles spaced 10 ft. and wheels with 6 ft. centers, and occupying a space 10 ft. wide and 30 ft. long, with 10 tons on rear axle and 5 tons on front axle, and with rear wheels 15 in. wide.

Loads for the trusses as per Table I. No bridge, however, to be designed for a load of less than 800 lb. per lineal foot of bridge.

Class E_1. For Heavy Electric Railways Only.—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are loaded with a load of 40,000 lb., making a total of 160,000 lb. Or a uniform load of 6,000 lb. per lineal foot for all spans up to 50 ft., reduced to 4,500 lb. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

Class E_2. For Medium Electric Railways Only.—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5-ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15-ft. centers. The axles are loaded with a load of 25,000 lb., making a total load of 100,000 lb. Or a uniform load of 3,500 lb. per lineal foot for all spans up to 50 ft., reduced to 2,000 lb. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

Class E_3. For Light Electric Railways Only.—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5-ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft.-centers. The axles are loaded with a load of 20,000 lb. making a total load of 80,000 lb. Or a uniform load of 2,500 lb. per lineal foot for all spans up to 50 ft., reduced to 1,500 lb. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

TABLE I.

LIVE LOADS FOR THE TRUSSES

Span in Feet.	Class A.		Class B.		Class C.		Class D_1.	Class D_2.
	Pounds per Lineal Foot of Each Car Track.	Pounds per Square Foot of Remaining Floor Surface.	Pounds per Lineal Foot of Each Car Track.	Pounds per Square Foot of Remaining Floor Surface.	Pounds per Lineal Foot of Each Car Track.	Pounds per Square Foot of Remaining Floor Surface.	Pounds per Square Foot of Floor Surface.	Pounds per Square Foot of Floor Surface.
Up to 30.......	1,800	125	1,800	125	1,800	125	125	100
80.......	1,800	105	1,800	85	1,200	85	85	75
160.......	1,440	88	1,440	68	1,080	68	68	60
200 and over	1,200	80	1,200	60	1,000	60	60	50
Loads for intermediate spans to be proportional.								

30. **Wind Loads.**—The lateral bracing in the unloaded chords of truss bridges shall be designed for a lateral wind load of 150 lb. per lineal foot of bridge, considered as a moving load. The lateral bracing in the loaded chords of truss bridges shall be designed for a lateral wind load of 300 lb. per lineal foot of bridge, considered as a moving load. For spans over 300 ft. each of the above loadings shall be increased 10 lb. for each 20 ft. increase in span. In highway bridges not carrying electric cars the end-posts of through and deck bridges and the intermediate posts of through bridges shall be designed for a combination (1) of the dead load stresses and the total live load stresses; or (2) of the dead load stresses, the live load stresses, the impact and centrifugal stresses, and one-half the total wind load stresses. In low truss bridges and plate girders not carrying electric cars the wind load on the unloaded chord may be omitted and the lateral bracing be designed for a lateral wind load of 300 lb. per lineal foot treated as a moving load. In bridges with sway bracing one-half of the wind load may be assumed to pass to the lower chord through the sway bracing.

End-posts of riveted through trusses with end floorbeams riveted rigidly at ends, shall be assumed as fixed at lower end.

31. In trestle towers the bracing and columns shall be designed to resist the following lateral forces, in addition to the stresses due to dead and live loads: The trusses loaded or unloaded, the lateral pressures specified above; and a lateral pressure of 100 lb. for each vertical lineal foot of trestle bent.

32. **Temperature.**—Stresses due to a variation in temperature of 150 degrees shall be provided for (§ 81).

33. **Centrifugal Force of Train.**—Structures located on curves shall be designed for the centrifugal force of the live load acting at the top of the rail. The centrifugal force shall be calculated by the following formula: $C = 0.03\ W \cdot D$; where C = centrifugal force in lb.; W = weight of train in lb.; and D = degree of curvature.

34. **Longitudinal Forces.**—The stresses produced in the bracing of the trestle towers, in any members of the trusses, or in the attachments of the girders or trusses to their bearings, by sud-

denly stopping the maximum electric car trains on any part of the work must be provided for; the coefficient of friction of the wheels on the rails being assumed as 0.20.

35. All parts shall be so designed that the stresses coming upon them can be accurately calculated.

PART III. UNIT STRESSES AND PROPORTION OF PARTS.

36. **Unit Stresses.**—All parts of the structure shall be proportioned so that the sum of the maximum stresses shall not exceed the following amounts in lb. per sq. in., except as modified by § 45 and § 48.

Impact.—The dynamic increment of the live load stress shall be added to the maximum live load stresses as follows:

For the floor and its supports including floor slabs, floor joist, floorbeams and hangers, 30 per cent.

For all truss members other than the floor and its supports, the impact increment shall be $I = 100/(L + 300)$, where L = length of span for simple highway spans (for trestle bents, towers, movable bridges, arch and cantilever bridges, and for bridges carrying electric trains, L shall be taken as the loaded length of the bridge in feet producing maximum stress in the member).

Impact shall not be added to the stresses produced by longitudinal, centrifugal and lateral or wind forces.

37. **Tension.**—Axial tension on net section 16,000

The lengths of riveted tension members in horizontal or inclined positions shall not exceed 200 times their radius of gyration about the horizontal axis. The horizontal projection of the unsupported portion of the member is to be considered as the effective length.

38. **Compression.**—Axial compression on gross section .... $16{,}000 - 70 \cdot l/r$ with a maximum of 14,000 lb.; where "l" is the length of member in inches and "r" is the least radius of gyration in inches.

No compression member, however, shall have a length exceeding 100 times its least radius of gyration for main members or 120 times for laterals for classes A, B, C, E_1, E_2, and E_3; or 125 times its least radius of gyration for main members or 150 times for laterals for classes D_1 and D_2.

39. **Bending.**—Bending: on extreme fibers of rolled shapes, built sections and girders;
net section 16,000
on cast iron 3,000
on extreme fibers of pins 24,000

40. **Shearing.**—Shearing: shop driven rivets and pins 12,000
field driven rivets and turned bolts 10,000
plate girder webs; gross section 10,000
cast iron 1,500

41. **Bearing.**—Bearing; shop driven rivets and pins 24,000
field driven rivets and turned bolts 20,000
cast iron 12,000
granite masonry and Portland cement concrete 600
sandstone and limestone 400
expansion rollers; per lineal inch .... $600d$
cast iron expansion rockers; per lineal inch .... $300d$
where "d" is the diameter of the roller in inches.

Rivets shall not be used in direct tension, except for lateral bracing where unavoidable: in which case the value for direct tension on the rivet shall be taken the same as for single shear.

42. **Alternate Stresses.**—Members subject to alternate stresses of tension and compression shall be proportioned for the stresses giving the largest section. If the alternate stresses occur in succession during the passage of one train, as in stiff counters, each stress shall be increased by 50 per cent of the smaller. The connections shall in all cases be proportioned for the sum of the stresses.

43. **Angles in Tension.**—When single-angle members subject to direct tension are fastened by one leg, only seventy-five per cent of the net area shall be considered effective. Angles with lug angle connections shall not be considered as fastened by both legs.

44. **Net Section.**—In members subject to tensile stresses full allowance shall be made for reduction of section by rivet-holes, screw-threads, etc. In calculating net area the rivet-holes shall be taken as having a diameter ⅛ in. greater than the normal size of rivet.

The net section of riveted members shall be the least area which can be obtained by deducting from the gross sectional area the areas of holes cut by any plane perpendicular to the axis of the member and parts of the areas of other holes on one side of the plane, within a distance of 4 inches, and which are on other gage lines than those of the holes cut by the plane, the parts being determined by the formula:

$$A(1 - p/4),$$

in which A = the area of the hole, and
p = the distance in inches of the center of the hole from the plane.

45. **Long Span Bridges.**—For long span bridges, where the ratio of the length to width of span is such that it makes the top chords acting as a whole, a longer column than the segments of the chords, the chord shall be proportioned for the greater length.

46. **Wind Stresses.**—The stresses in truss members or trestle posts from assumed wind forces need not be considered except as follows:

1. When the direct wind stresses per square inch in any member exceed 25 per cent of the stresses due to dead and live loads in the same member. The section shall then be increased until the total unit stress shall not exceed by more than 25 per cent the maximum allowable stress for dead and live loads.
2. When the wind stress alone or in combination with a possible temperature stress can neutralize or reverse the stresses in the member.

When both direct and flexural stresses due to wind are considered 50 per cent may be added to allowable stresses for dead and live loads, provided the area thus obtained is not less than required for dead and live loads alone, or for dead, live and direct wind loads designed as in § 46.

47. **Combined Stresses.**—Members subjected to direct and bending stresses shall be designed so that the greatest fiber stress shall not exceed the allowable unit stress on the member.

48. **Stress Due to Weight and Eccentric Loading.**—If the fiber stress due to weight and eccentric loading on any member exceeds 10 per cent of the allowable unit stress on the member, such excess must be considered in proportioning the member. See § 46.

49. **Counters.**—Counters in bridges carrying electric cars shall be designed so that an increase of the live load of 25 per cent will not increase the stress in the counters more than 25 per cent.

50. **Design of Plate Girders.**—Plate girders shall be proportioned either by the moment of inertia of their net section; or by assuming that the flanges are concentrated at their centers of gravity, in which case one-eighth of the gross section of the web, if properly spliced, may be used as flange section. The thickness of web plates shall be not less than $\frac{5}{16}$ in., nor less than 1/160 of the unsupported distance between flange angles.

Compression Flanges.—In beams and plate girders the compression flanges shall have the same gross section as the tension flanges. Through plate girders shall have their top flanges stayed at each end of every floorbeam, or in case of solid floors, at distances not exceeding 12 ft., by knee braces or gusset plates. The stress per sq. in. in compression flange of any beam or girder shall not exceed $16,000 - 150\ l/b$, where l = unsupported distance and b = width of flange.

51. **Web Stiffeners.**—There shall be web stiffeners, generally in pairs, over bearings, at points of concentrated loading, and at other points where the thickness of the web is less than $\frac{1}{60}$ of the unsupported distance between flange angles. The distance between stiffeners shall not exceed that given by the following formula, with a maximum limit of six feet (and not greater than the clear depth of the web): $d = t\,(12,000 - s)/40$.

Where d = clear distance, between stiffeners of flange angles; t = thickness of web; s = shear in lb. per sq. in.

The stiffeners at ends and at points of concentrated loads shall be proportioned by the formula of paragraph 38, the effective length being assumed as one-half the depth of girders. End stiffeners and those under concentrated loads shall be on fillers and have their outstanding legs as wide as the flange angles will allow and shall fit tightly against them. Intermediate stiffeners may be offset or on fillers, and their outstanding legs shall be not less than one-thirtieth of the depth of girder, plus 2 in.

52. **Flange Rivets.**—The flanges of plate girders shall be connected to the web with a sufficient number of rivets to transfer the total shear at any point in a distance equal to the effective depth of the girder at that point combined with any load that is applied directly on the flange. The wheel loads, where the ties rest on the flanges, shall be assumed to be distributed over three ties.

53. **Depth Ratios.**—Trusses shall preferably have a depth of not less than one-tenth of the span. Plate girders and rolled beams, used as girders, shall preferably have a depth of not less than one-twelfth of the span. If shallower trusses, girders or beams are used, the section shall be increased so that the maximum deflection will not be greater than if the above limiting ratios had not been exceeded. For steel joists and track stringers, see § 19.

54. **Low Trusses.**—Riveted low trusses shall have top chords composed of a double web member with cover plate. The top chords shall be stayed against lateral bending by means of brackets or knee braces rigidly connected to the floorbeam at intervals not greater than twelve times the width of the cover plate. The posts shall be solid web members. The floorbeams shall be riveted, preferably above the lower chord. Pin-connected low truss bridges shall not be used.

55. **Rolled Beams.**—Rolled beams shall be designed by using their moments of inertia. The webs of rolled beams and plate girders shall be assumed to take all the shear.

PART IV. DETAILS OF DESIGN

General Requirements.

56. **Open Sections.**—Structures shall be so designed that all parts will be accessible for inspection, cleaning and painting.

57. **Water Pockets.**—Pockets or depressions which would hold water shall have drain holes, or be filled with waterproof material.

58. **Symmetrical Sections.**—Main members shall be so designed that the neutral axis will be as nearly as practicable in the center of section, and the neutral axes of intersecting main members of trusses shall meet at a common point.

59. **Counters.**—Rigid counters are preferred; and where subject to reversal of stress shall have riveted connections to the chords. Adjustable counters shall have open turn-buckles.

60. **Strength of Connections.**—The strength of connections shall be sufficient to develop the full strength of the member, even though the computed stress is less, the kind of stress to which the member is subjected being considered.

61. **Minimum Thickness.**—The minimum thickness of metal shall be $\frac{5}{16}$ in. in classes A, B, C, E_1, E_2 and E_3, except for fillers; and $\frac{1}{4}$ in. in classes D_1 and D_2, except for fillers and webs of channels. Webs of channels for classes D_1 and D_2 may have a minimum thickness of 0.20 in. The minimum angle shall be 2 in. × 2 in. × $\frac{1}{4}$ in. The minimum rod shall have an area of at least 1 sq. in., in all classes except D_1 and D_2, which shall have no rods less than $\frac{3}{4}$ in. in diameter. Webs of plate girders shall not be less than $\frac{5}{16}$ in.

62. **Pitch of Rivets.**—The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3 in. for $\frac{7}{8}$-in. rivets, $2\frac{1}{2}$ in. for $\frac{3}{4}$-in rivets, and 2 in. for $\frac{5}{8}$-in. rivets. The maximum pitch in the line of stress for members composed of plates and shapes shall be 16 times the thickness of the thinnest outside plate or 6 in. For angles with two gage lines and rivets staggered, the maximum shall be twice the above in each line. Where two or more plates are used in contact, rivets not more than 12 in. apart in either direction shall be used to hold the plates well together. In tension members composed of two angles in contact, a pitch of 12 in. will be allowed for riveting the angles together.

63. **Edge Distance.**—The minimum distance from the center of any rivet hole to a sheared edge shall be $1\frac{1}{2}$ in. for $\frac{7}{8}$-in. rivets, $1\frac{1}{4}$ in. for $\frac{3}{4}$-in. rivets, and $1\frac{1}{8}$ in. for $\frac{5}{8}$-in. rivets, and to a rolled edge $1\frac{1}{4}$, $1\frac{1}{8}$ and 1 in., respectively. The maximum distance from any edge shall be eight times the thickness of the plate, but shall not exceed 6 in.

64. **Maximum Diameter.**—The diameter of the rivets in any angle carrying calculated stress shall not exceed one-quarter the width of the leg in which they are driven. In minor parts $\frac{7}{8}$-in. rivets may be used in 3-in. angles, $\frac{3}{4}$-in. rivets in $2\frac{1}{2}$-in. angles, and $\frac{5}{8}$-in. rivets in 2-in. angles.

65. **Long Rivets.**—Rivets carrying calculated stress and whose grip exceeds four diameters shall be increased in number at least one per cent for each additional $\frac{1}{16}$-in. of grip.

66. **Pitch at Ends.**—The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets, for a length equal to one and one-half times the maximum width of member.

67. **Compression Members.**—In compression members the metal shall be concentrated as much as possible in webs and flanges. The thickness of each web shall be not less than one-thirtieth of the distance between its connections to the flanges. Cover plates shall have a thickness not less than one-fortieth of the distance between rivet lines.

68. **Minimum Angles.**—Flanges of girders and built members without cover plates shall have a minimum thickness of one-twelfth of the width of the outstanding leg.

69. **Batten Plates.**—The open sides of all compression members shall be stayed by batten plates at the ends and diagonal lattice-work at intermediate points. The batten plates must be placed as near the ends as practicable, and shall have a length not less than the greatest width of the member or $1\frac{1}{2}$ times its least width.

70. **Lacing Bars.**—The lacing of compression members shall be proportioned to resist a shearing stress of $2\frac{1}{2}$ per cent of the direct stress. The minimum width of lacing bars shall be $1\frac{3}{4}$ in. for members 6 in. in width, 2 in. for members 9 in. in width, $2\frac{1}{4}$ in. for members 12 in. in width, $2\frac{1}{2}$ in. for members 15 in. in width, nor 3 in. for members 18 in. and over in width. Single lacing bars shall have a thickness not less than one-fortieth, or double lacing bars connected by a rivet at the intersection, not less than one-sixtieth of the distance between the rivets connecting them

to the members. They shall be inclined at an angle not less than 60° to the axis of the member for single lacing, nor less than 45° for double lacing with riveted intersections.

71. **Spacing of Lacing Bars.**—Lacing bars shall be so spaced that the portion of the flange included between their connection shall be as strong as the member as a whole. The pitch of the lacing bars must not exceed the width of the channel plus nine inches.

72. **Rivets in Flanges.**—Five-eighths-inch rivets shall be used for lacing flanges less than 2½ in. wide; ¾-in. for flanges from 2½ to 3½ in. wide; ⅞-in. rivets shall be used in flanges 3½ in. and over. Lacing bars with two rivets shall be used for flanges over 5 in. wide.

73. **Splices.**—In compression members joints with abutting faces planed shall be placed as near the panel points as possible, and must be spliced on all sides with at least two rows of rivets on each side of the joint. Joints with abutting faces not planed shall be fully spliced. Joints in tension members shall be fully spliced.

74. **Pin Plates.**—Where necessary, pin-holes shall be reinforced by plates, some of which must be of the full width of the member, so the allowed pressure on the pins shall not be exceeded, and so the stresses shall be properly distributed over the full cross-section of the members. These reinforcing plates must contain enough rivets to transfer their proportion of the bearing pressure, and at least one plate on each side shall extend not less than 6 in. beyond the edge of the nearest batten plate.

75. **Riveted Tension Members.**—Riveted tension members shall have an effective section through the pin-holes 25 per cent in excess of the net section of the member, and back of the pin at least 75 per cent of the net section through the pin-hole.

76. **Pins.**—Pins shall be long enough to insure a full bearing of all the parts connected upon the turned body of the pin. The diameter of the pin shall not be less than ¾ of the depth of any eye-bar attached to it. They shall be secured by chambered Lomas nuts or be provided with washers if solid nuts are used. The screw ends shall be long enough to admit of burring the threads.

77. **Filling Rings.**—Members packed on pins shall be held against lateral movement.

78. **Bolts.**—Where members are connected by bolts, the turned body of these bolts shall be long enough to extend through the metal. A washer at least ¼ in. thick shall be used under the nut. Bolts shall not be used in place of rivets except by special permission. Heads and nuts shall be hexagonal.

79. **Indirect Splices.**—Where splice plates are not in direct contact with the parts which they connect, rivets shall be used on each side of the joint in excess of the number theoretically required to the extent of one-third of the number for each intervening plate.

80. **Fillers.**—Rivets carrying stress and passing through fillers shall be increased 50 per cent in number; and the excess rivets, when possible, shall be outside of the connected member.

81. **Expansion.**—Provision for expansion to the extent of ⅛ in. for each 10 ft. shall be made for all bridge structures. Efficient means shall be provided to prevent excessive motion at any one point (§ 32).

82. **Expansion Bearings.**—Spans of 60 ft. and over resting on masonry shall have turned rollers or rockers at one end; and those of less length shall be arranged to slide on smooth metal surfaces.

83. **Fixed Bearings.**—Movable bearings shall be designed to permit motion in one direction only. Fixed bearings shall be firmly anchored to the masonry (§ 87).

84. **Rollers.**—Expansion rollers shall be not less than 3 in. in diameter for spans of 100 feet or less, and shall be increased 1 in. for each 100 ft. additional. They shall be coupled together with substantial side bars, which shall be so arranged that the rollers can be readily cleaned.

85. **Bolsters.**—Bolsters or shoes shall be so constructed that the load will be distributed over the entire bearing.

86. **Pedestals and Bed Plates.**—Built pedestals shall be made of plates and angles. All bearing surfaces of the base plates and vertical webs must be planed. The vertical webs must be secured to the base by angles having two rows of rivets in the vertical legs. No base plate or web connecting angle shall be less in thickness than ⅜ in. The vertical webs shall be of sufficient height and must contain material and rivets enough to practically distribute the loads over the bearings or rollers.

Where the size of the pedestal permits, the vertical webs must be rigidly connected transversely.

The details of cast iron or cast steel shoes shall be subject to the special approval of the engineer. The vertical webs of cast iron rockers and pedestals shall be designed for an allowable unit stress of $9{,}000 - 40l/r$, where h = height and r = radius of gyration of vertical web, both in inches.

87. All the bed-plates and bearings under fixed and movable ends must be fox-bolted to the masonry; for trusses, these bolts must not be less than $1\frac{1}{4}$ in. diameter; for plate and other girders, not less than $\frac{7}{8}$ in. diameter.

88. **Wall Plates.**—Wall plates may be cast or built up; and shall be so designed as to distribute the load uniformly over the entire bearing. They shall be secured against displacement.

89. **Anchorage.**—Anchor bolts for viaduct towers and similar structures shall be long enough to engage a mass of masonry the weight of which is at least one and one-half times the uplift (§ 11).

90. **Inclined Bearings.**—Bridges on an inclined grade without pin shoes shall have the sole plates beveled so that the masonry and expansion surfaces may be level.

91. **Camber.**—Truss spans shall be given a camber by making the panel length of the top chords, or their horizontal projections, longer than the corresponding panels of the bottom chord in the proportion of $\frac{3}{16}$ in. in 10 ft. Plate girder spans need not be cambered.

92. **Eye-bars.**—The eye-bars composing a member shall be so arranged that adjacent bars shall not have their surfaces in contact; they shall be as nearly parallel to the axis of the truss as possible, the maximum inclination of any bar being limited to one inch in 16 ft.

PART V. MATERIALS AND WORKMANSHIP.

MATERIAL.

93. **Process of Manufacture.**—Steel shall be made by the open-hearth process and shall comply with the standard specifications for structural steel for bridges adopted by the American Society for Testing Materials.

(Sections 94 to 117 inclusive cover the American Society for Testing Materials Specifications for Steel for Bridges, see Ketchum's Structural Engineer's Handbook).

118. **Timber.**—The timber shall be strictly first-class spruce, white pine, Douglas fir, Southern yellow pine, or white oak bridge timber; sawed true and out of wind, full size, free from wind shakes, large or loose knots, decayed or sapwood, wormholes or other defects impairing its strength or durability.

WORKMANSHIP.

119. **General.**—All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works.

120. **Straightening Material.**—Material shall be thoroughly straightened in the shop, by methods that will not injure it, before being laid off or worked in any way.

121. **Finish.**—Shearing shall be neatly and accurately done and all portions of the work exposed to view neatly finished.

122. **Size of Rivets.**—The size of rivets, called for on the plans, shall be understood to mean the actual size of the cold rivet before heating.

123. **Rivet Holes.**—When general reaming is not required the diameter of the punch shall not be more than $\frac{1}{16}$ in. greater than the diameter of the rivet; nor the diameter of the die more than $\frac{1}{8}$ in. greater than the diameter of the punch. Material more than $\frac{3}{4}$ in. thick shall be sub-punched and reamed or drilled from the solid.

124. **Punching.**—All punching shall be accurately done. Drifting to enlarge unfair holes will not be allowed. If the holes must be enlarged to admit the rivet, they shall be reamed. Poor matching of holes will be cause for rejection.

125. **Sub-punching and Reaming.**—Where reaming is required, the punch used shall have a diameter not less than $\frac{3}{16}$ in. smaller than the nominal diameter of the rivet. Holes shall then be reamed to a diameter not more than $\frac{1}{16}$ in. larger than the nominal diameter of the rivet. All reaming shall be done with twist drills. (§ 140).

126. **Reaming After Assembling.**—When general reaming is required it shall be done after the pieces forming one built member are assembled and firmly bolted together. If necessary to take the pieces apart for shipping and handling, the respective pieces reamed together shall be so marked that they may be reassembled in the same position in the final setting up. No interchange of reamed parts will be allowed.

127. **Edge Planing.**—Sheared edges or ends shall, when required, be planed at least $\frac{1}{8}$ in.

128. **Burrs.**—The outside burrs on reamed holes shall be removed.

129. **Assembling.**—Riveted members shall have all parts well pinned up and firmly drawn together with bolts, before riveting is commenced. Contact surfaces to be painted.

130. **Lacing Bars.**—Lacing bars shall have neatly rounded ends, unless otherwise called for.

131. **Web Stiffeners.**—Stiffeners shall fit neatly between flanges of girders. Where tight fits are called for, the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

132. **Splice Plates and Fillers.**—Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$ in. of flange angles.

133. **Web Plates.**—Web plates of girders, which have no cover plates, shall be flush with the backs of angles or project above the same not more than $\frac{1}{8}$ in., unless otherwise called for. When web plates are spliced, not more than $\frac{1}{4}$ in. clearance between ends of plates will be allowed.

134. **Connection Angles.**—Connection angles for floorbeams and stringers shall be flush with each other and correct as to position and length of girder. In case milling (of all such angles) is needed or is required after riveting, the removal of more than $\frac{1}{16}$ in. from their thickness will be cause for rejection.

135. **Rivets.**—Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

136. **Riveting.**—Rivets shall look neat and finished, with heads of approved shape, full and of equal size. They shall be central on shank and grip the assembled pieces firmly. Recupping and calking will not be allowed. Loose, burned or otherwise defective rivets shall be cut out and replaced. In cutting out rivets, great care shall be taken not to injure the adjacent metal. If necessary, they shall be drilled out.

137. **Turned Bolts.**—Wherever bolts are used in place of rivets which transmit shear, the holes shall be reamed parallel and the bolts turned to a driving fit. A washer not less than $\frac{1}{4}$ in. thick shall be used under nut.

138. **Members to be Straight.**—The several pieces forming one built member shall be straight and fit closely together, and finished members shall be free from twists, bends or open joints.

139. **Finish of Joints.**—Abutting joints shall be cut or dressed true and straight and fitted close together, especially where open to view. In compression joints, depending on contact bearing, the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

140. **Field Connections.**—Holes for floorbeam and stringer connections shall be sub-punched and reamed according to paragraph 125, to a steel templet one inch thick. (If required, all other field connections, except those for laterals and sway bracing, shall be assembled in the shop and the unfair holes reamed; and when so reamed, the pieces shall be match-marked before being taken apart.)

141. **Eye-bars.**—Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body when tested to rupture. The thickness of head and neck shall not vary more than $\frac{1}{16}$ in. from that specified.

142. **Boring Eye-bars.**—Before boring, each eye-bar shall be properly annealed and carefully straightened. Pin-holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that, when placed together, pins $\frac{1}{32}$ in. smaller in diameter than the pin-holes can be passed through the holes at both ends of the bars at the same time without forcing.

143. **Pin-Holes.**—Pin-holes shall be bored true to gages, smooth and straight; at right angles to the axis of the member and parallel to each other, unless otherwise called for. The boring shall be done after the member is riveted up.

144. **Variation in Pin-Holes.**—The distance center to center of pin-holes shall be correct within $\frac{1}{32}$ in., and the diameter of the holes not more than $\frac{1}{50}$ in. larger than that of the pin, for pins up to 5-in. diameter, and $\frac{1}{32}$ in. for larger pins.

145. **Pins and Rollers.**—Pins and rollers shall be accurately turned to gages and shall be straight and smooth and entirely free from flaws.

146. **Screw Threads.**—Screw threads shall make tight fits in the nuts and shall be U. S. standard, except above the diameter of $1\frac{3}{8}$ in., when they shall be made with six threads per inch.

147. **Annealing.**—Steel, except in minor details, which has been partially heated, shall be properly annealed.

148. **Steel Castings.**—All steel castings shall be annealed.

149. **Welds.**—Welds in steel will not be allowed except to remedy minor defects in steel castings.

150. **Bed Plates.**—Expansion bed plates shall be planed true and smooth. Cast wall plates shall be planed top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

151. **Pilot Nuts.**—Pilot and driving nuts shall be furnished for each size of pin, in such numbers as may be ordered.

152. **Field Rivets.**—Field rivets shall be furnished to the amount of 15 per cent plus ten rivets in excess of the nominal number required for each size.

153. **Shipping Details.**—Pins, nuts, bolts, rivets and other small details shall be boxed or crated.

154. **Weight.**—The weight of every piece and box shall be marked on it in plain figures.

155. **Weight Paid For.**—The payment for pound price contracts shall be based on scale weights of the metal in the fabricated structure, including field rivets 15 per cent plus 10 rivets in excess of the number nominally required. The weight of the shop coat of paint, field paint, cement, fitting up bolts, pilot nuts, driving caps, boxes and barrels used for packing, and material used in supporting members on cars shall be excluded. If the scale weight is more than 2½ per cent under the computed weight it may be cause for rejection. The greatest allowable variation of the total scale weight of any structure from the weights computed from the approved shop drawings shall be 1½ per cent. Any weight in excess of 1½ per cent above the computed weight shall not be paid for. The weights of rolled shapes and plates up to and including 36 in. in width shall be computed on the basis of their normal weights and dimensions, as shown on the approved drawings, deducting for all copes, cuts and open holes. With plates more than 36 in. in width, the weights are to be calculated in the same manner as for plates 36 in. and under, except that one-half the percentage of overrun given in the Standard Specifications for Structural Steel for Bridges of the American Society for Testing Materials shall be added. The weight of heads of shop driven rivets shall be included in the computed weight. The weights of castings shall be computed from the dimensions shown on the approved drawings, with an addition of 10 per cent for fillets and overrun.

SHOP PAINTING.

156. **Cleaning.**—Steel work, before leaving the shop, shall be thoroughly cleaned and given one good coating of pure linseed oil, or such paint as may be called for, well worked into all joints and open spaces.

157. **Contact Surfaces.**—In riveted work, the surfaces coming in contact shall each be painted before being riveted together.

158. **Inaccessible Surfaces.**—Pieces and parts which are not accessible for painting after erection, including tops of stringers, eye-bar heads, ends of posts and chords, etc., shall have a good coat of paint before leaving the shop.

159. **Condition of Surfaces.**—Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

160. **Machine-finished Surfaces.**—Machine-finished surfaces shall be coated with white lead and tallow before shipment or before being put out into the open air.

INSPECTION AND TESTING AT THE SHOP AND MILL.

161. **Facilities for Shop Inspection.**—The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of workmanship at the shop where material is manufactured. He shall furnish a suitable testing machine for testing full-sized members, if required.

162. **Starting Work in Shop.**—The purchaser shall be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect material and workmanship.

163. **Copies of Mill Orders.**—The purchaser shall be furnished complete copies of mill orders, and no material shall be rolled, nor work done, before the purchaser has been notified where the orders have been placed, so that he may arrange for the inspection.

164. **Facilities for Mill Inspection.**—The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of all material at the mill where it is manufactured. He shall furnish a suitable testing machine for testing the specimens, as well as prepare the pieces for the machine, free of cost.

165. **Access to Mills.**—When an inspector is furnished by the purchaser to inspect material at the mills, he shall have full access, at all times, to all parts of mills where material to be inspected by him is being manufactured.

166. **Access to Shop.**—When an inspector is furnished by the purchaser, he shall have full access, at all times, to all parts of the shop where material under his inspection is being manufactured.

167. **Accepting Material or Work.**—The inspector shall stamp each piece accepted with a private mark. Any piece not so marked may be rejected at any time, and at any stage of the work. If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the purchaser.

168. **Shop Plans.**—The purchaser shall be furnished complete shop plans (§ 13).

169. **Shipping Invoices.**—Complete copies of shipping invoices shall be furnished to the purchaser with each shipment.

FULL-SIZED TESTS.

170. **Test to Prove Workmanship.**—Full-sized tests on eye-bars and similar members, to prove the workmanship, shall be made at the manufacturer's expense, and shall be paid for by the purchaser at contract price, if the tests are satisfactory. If the tests are not satisfactory, the members represented by them will be rejected.

171. **Eye-bar Tests.**—In eye-bar tests, the fracture shall be silky, the elongation in 10 ft., including the fracture, shall be not less than 15 per cent; and the ultimate strength and true elastic limit shall be recorded (§ 141).

ERECTION.

172. If the contractor erects the bridge he shall, unless otherwise specified, furnish all staging and falsework, erect and adjust all metal work, and shall frame and put in place all floor timbers, guard timbers, trestle timbers, etc., complete ready for traffic.

The contractor shall put in place all stone bolts and anchors for attaching the steel work to the masonry. He shall drill all the necessary holes in the masonry, and set all bolts with neat Portland cement.

173. Field rivets shall preferably be driven by pneumatic riveters of approved make. A pneumatic bucker shall be used with a pneumatic riveter. Splices and field connections shall have 50 per cent of the holes filled with bolts and drift pins (of which one-fifth shall be drift pins) before riveting. Splices and connections carrying traffic during erection shall have 75 per cent of the holes so filled. Rivets in splices of compression chords shall not be driven until the abutting surfaces have been brought into contact throughout, and submitted to full dead load stress. Field riveting shall be done to the satisfaction of the engineer.

The fence may be field bolted, all other connections shall be field riveted.

174. The erection will also include all necessary hauling from the railroad station, the unloading of the materials and their proper care until the erection is completed.

175. Whenever new structures are to replace existing ones, the latter are to be carefully taken down and removed by the contractor to some place where the material can be hauled away.

176. The contractor shall so conduct his work as not to interfere with traffic, interfere with the work of other contractors, or close any thoroughfare on land or water.

177. The contractor shall assume all risks of accidents and damages to persons and properties prior to the acceptance of the work.

178. The contractor must remove all falsework, piling and other obstructions or unsightly material produced by his operations.

PAINTING AFTER ERECTION.

179. After the bridge is erected the metal work shall be thoroughly cleaned of mud, grease or other material, then be thoroughly and evenly painted with two coats of paint of the kind specified by the engineer, mixed with linseed oil. All recesses which may retain water, or through which water can enter, must be filled with thick paint or some waterproof cement before the final painting. The different coats of paint must be of distinctly different shades or colors, and one coat must be allowed to dry thoroughly before the second coat is applied. All painting shall be done with round brushes of the best quality obtainable on the market. The paint shall be delivered on the work in the manufacturer's original packages and be subject to inspection. If tests made by the inspector shows that the paint is adulterated, the paint will be rejected and the contractor shall pay the cost of the analyses, and shall scrape off and thoroughly clean and repaint all material that has been painted with the condemned paint. The paint shall not be thinned with anything whatsoever; in cold weather the paint may be thinned by heating under the direction of the inspector. No turpentine nor benzine shall be allowed on the work, except by the permission of the inspector, and in such quantity as he shall allow. The inspector shall be notified when any painting is to be done by the contractor, and no painting shall be done until the inspector has approved the surface to which the paint is to be applied. Paint shall not be applied out of doors in freezing, rainy, or misty weather, and all surfaces to which paint is to be applied shall be dry, clean and warm. In cool weather the paint may be thinned by heating, and this may be required by the inspector.

REFERENCES.—For the calculation of stresses in bridge trusses and plate girders, for details of bridges, for the design of bridge details, and for additional examples of highway bridges, see the author's "The Design of Highway Bridges of Steel, Timber and Concrete," Second Edition, 1920.

CHAPTER IV.

STEEL RAILWAY BRIDGES.

TYPES OF STEEL BRIDGES.—The same types of trusses are used for railway as for highway bridges, Fig. 4, Chapter III. Beam bridges are used for short spans, and plate girders up to spans of about 125 ft. Riveted truss spans are used for spans of 100 ft. and upwards. Pin-connected truss spans are still used for long span bridges and by a few railroads for spans of 150 ft. and upwards. Many railroads are building riveted trusses for spans of more than 200 ft., and riveted truss spans of 300 ft. are not uncommon. The new terminal bridge over the Missouri River at Kansas City, Mo., has riveted trusses with a span of 425 ft. 6½ in. The Norfolk & Western R. R. has constructed a double track bridge over the Ohio River with a span of 520 ft., which is riveted with the exception of four bottom chord panel points, which have pin joints. The lengths and types of railway bridges as used by different railroads are given in Table XII in the latter part of this chapter. The longest simple truss span is 668 ft. and is in the Municipal Bridge over the Mississippi River at St. Louis, Mo. The maximum practical length of simple span truss bridges made of carbon steel is about 550 feet; while with nickel steel it is practical to build simple truss spans up to 750 feet and economical to build simple truss spans up to 700 feet. The proposed Metropolis Bridge over the Ohio River will be a double track simple truss bridge with a span of 720 feet.

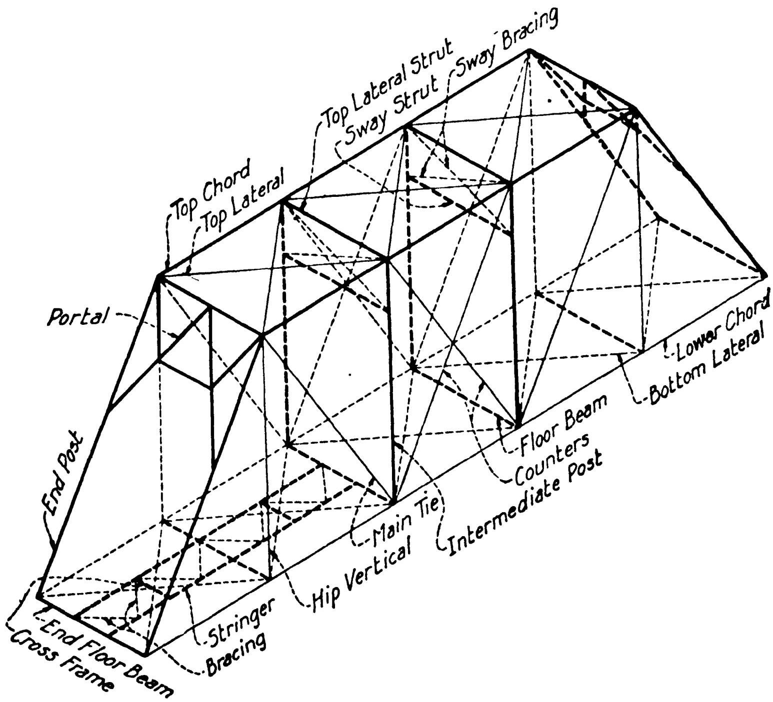


FIG. 1. DIAGRAMMATIC SKETCH OF A RAILWAY TRUSS BRIDGE.

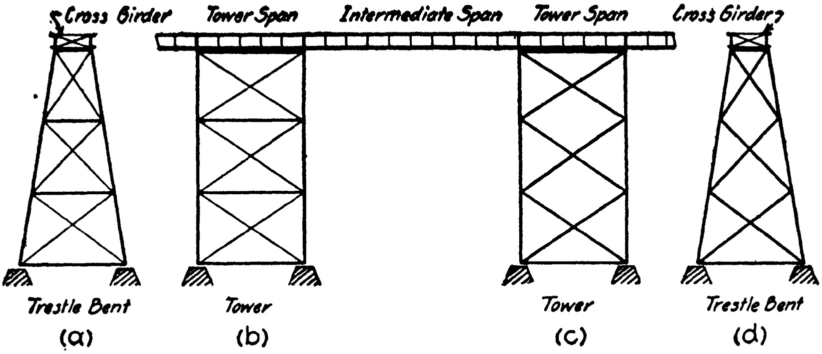


FIG. 2. RAILWAY STEEL TRESTLE.

TABLE I.

DATA ON RAILROAD BRIDGES DESIGNED UNDER COMMON STANDARD (HARRIMAN LINES) SPECIFICATIONS C. S. 1006.

SINGLE TRACK BRIDGES.					DOUBLE TRACK BRIDGES.				
Length of Span, Ft.	Distance Center to Center of Trusses or Girders, Ft.-In.	Dist. C. to C. of Chords or B. to B. of Angles, Ft.-In.	No. of Panels.	Total Weight, Lb.	Length of Span, Ft.	Distance Center to Center of Trusses or Girders, Ft.-In.	Dist. C. to C. of Chords or B. to B. of Angles, Ft.-In.	No. of Panels.	Total Weight, Lb.
	Through Plate Girders					Through Plate Girders			
30	13-6	4- 0½	3	27,500	50	29-6	8-0½	4	142,000
40	15-6	5- 0½	4	41,900	60	29-6	9-0½	5	173,000
50	15-6	5- 8½	5	56,600	70	29-6	9-6½	6	221,000
60	16-0	6- 4½	6	79,600	80	30-0	10-0½	7	277,000
70	16-6	7- 0½	7	105,100	90	30-0	10-6½	8	317,200
80	16-6	8- 0½	8	132,300					
90	16-6	8- 6½	9	161,350					
100	16-6	9- 0½	10	198,500					
	Deck Plate Girder					Through Rivet Span			
20	7-0	1- 8	3	12,800	100	30-6	30-0	4	360,000
30	7-0	4- 0½	4	14,900	110	30-6	30-0	4	400,000
40	7-0	4-11¾	8	23,800	125	30-6	31-0	5	472,600
50	7-0	5-11¾	8	34,300	140	. . .	. . . .	.	. . . . . .
60	7-0	6- 5¾	10	47,500					
70	8-0	8- 3¾	10	68,000					
80	8-0	8- 8¾	10	87,800					
90	9-0	9- 1¾	12	113,200					
100	9-0	9- 3¾	12	137,800					
	Through Rivet Span					Through Pin Span			
100	16-6	29- 0	4	165,000	150	30-6	33-0	6	633,000
110	16-6	29- 0	4	185,000	160	. . . .	. . . .	.	. . . . . .
125	16-6	30- 0	5	220,000	180	. . . .	. . . .	.	. . . . . .
140	17-0	31- 0	6	273,000	200	30-6	40-0	7	932,200
150	17-0	31- 0	6	311,000					
	Through Pin Span								
150	17-0	31- 0	6	304,000					
160	17-0	32- 0	6	348,000					
180	17-0	33- 0	7	417,000					
200	17-0	32-& 38	7	485,000					

A diagramatic sketch of a truss railway bridge is shown in Fig. 1. The names of the different members are shown on the diagram. The floor may be carried on two or more stringers. Two stringers are commonly used for an open timber floor and two or four stringers for a ballasted floor.

A railway steel trestle is shown in Fig. 2. Steel trestles are commonly built with the intermediate spans equal to twice the tower spans; 60 feet and 30 feet, and 80 feet and 40 feet being common lengths of span.

Swing, movable, cantilever and suspension bridges will not be considered in this chapter.

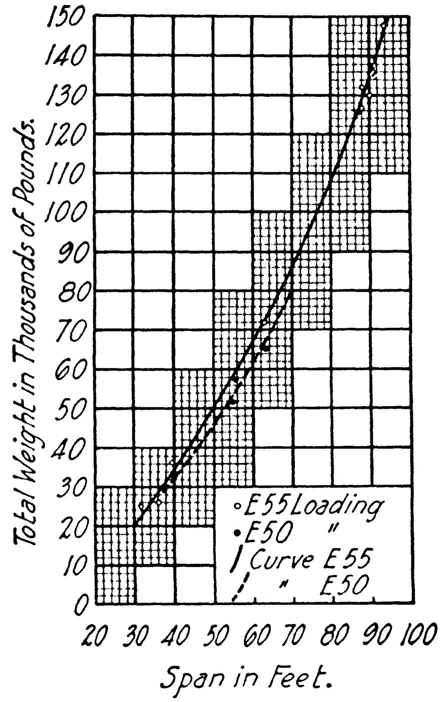


FIG. 3. WEIGHT OF SINGLE TRACK DECK PLATE GIRDER SPANS, CONCRETE BALLAST FLOOR. CHICAGO, MILWAUKEE & ST. PAUL RY.

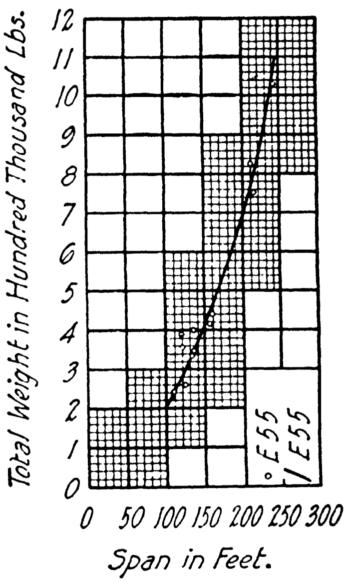


FIG. 4. WEIGHT OF SINGLE TRACK RIVETED DECK TRUSS SPANS. CHICAGO, MILWAUKEE & ST. PAUL RY.

WEIGHTS OF RAILWAY BRIDGES.—The weights of railway bridges vary with the loading, the specifications, the span, the width, the type of floor, and with the design. The weights of the total structural steel in single track bridges of different types as designed and built by the Chicago, Milwaukee & St. Paul Ry. are given in Fig. 3 to Fig. 10, inclusive.

Weights of single track plate girder spans as designed and built by the Illinois Central Railroad are given in Fig. 11, Fig. 12 and Fig. 13; weights of single track through bridges are given in Fig. 14, weights of signal bridges are given in Fig. 15, and weights of single track draw spans are given in Fig. 16. Weights and other data for railway bridges designed by the Harriman Lines, under "Common Standard Specification 1006" (approximately equal to Cooper's E 55), are given in Table I.

Weights of single track steel viaducts as designed by the McClintic-Marshall Construction Co. are given in Fig. 17.

For the relative weights of railway bridges built of carbon and of nickel steel, see paper entitled "Nickel Steel for Bridges," by Mr. J. A. L. Waddell, M. Am. Soc. C. E., printed in Trans. Am. Soc. C. E., Vol. 63, 1909.

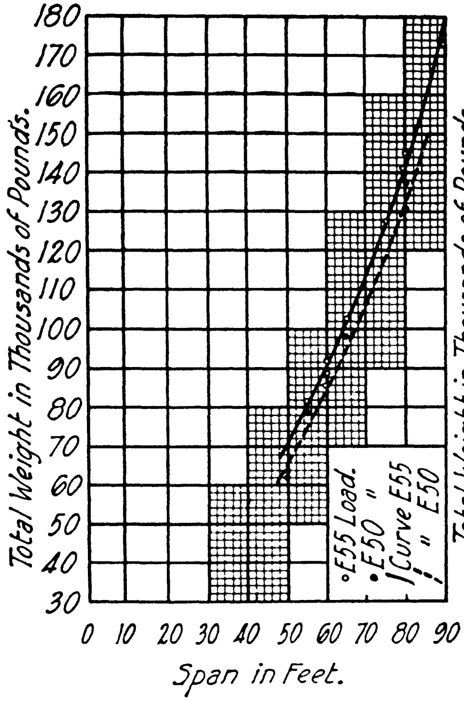


FIG. 5. WEIGHT OF SINGLE TRACK THROUGH PLATE GIRDER SPANS. TYPE C4 (FLANGES OF 2 ANGLES AND COVER PLATES, TWO STRINGERS). CHICAGO, MILWAUKEE & ST. PAUL RY.

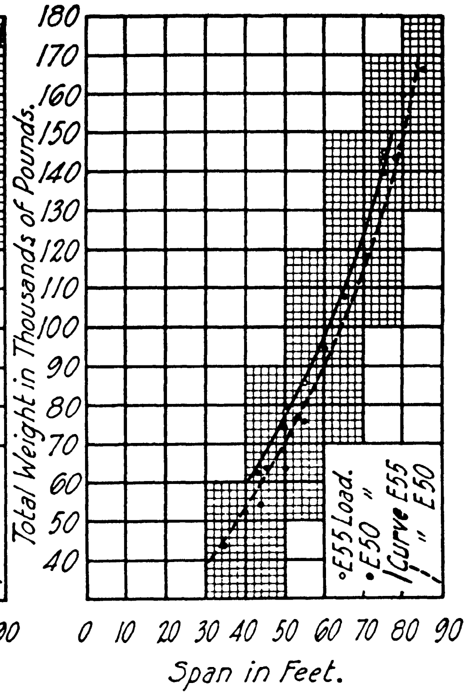


FIG. 6. WEIGHT OF THROUGH PLATE GIRDER SPANS. TYPE C3 (FLANGES OF 2 ANGLES AND COVER PLATES, SHALLOW FLOOR, 4 STRINGERS). CHICAGO, MILWAUKEE & ST. PAUL RY.

LOADS.*—The dead load of a railway bridge is assumed to act at the joints the same as in a highway bridge. The dead joint loads are commonly assumed to act on the loaded chord, but may be assumed as divided between the panel points of the two chords, one-third and two-thirds of the dead loads usually being assumed as acting at the panel points of the unloaded and the loaded chords, respectively, see discussion of specifications in the last part of this chapter.

The live load on a railway bridge consists of wheel loads, the weights and spacing of the wheels depending upon the type of the rolling stock used. The locomotives and cars differ so much that it would be difficult if not impossible to design the bridges on any railway system for the actual conditions, and conventional systems of loading, which approximate the actual conditions, are assumed. The conventional systems for calculating the live load stresses in railway bridges that have been most favorably received are: (1) Cooper's Conventional System of Wheel Concentrations; (2) the use of an Equivalent Uniform Load; and (3) the use of a uniform load and one or two wheel concentrations. In addition to these some railroads specify special engine loadings. The three methods will be briefly described.

* For live loads for railway bridges in 1924, see latter part of this chapter.

Cooper's Conventional System of Wheel Concentrations.—In Cooper's loadings two consolidation locomotives are followed by a uniformly distributed train load. The typical loading for Cooper's Class E 40, E 45, E 50, E 55 and E 60, are shown in Fig. 18. The loads on the drivers in thousands of pounds and the uniform train load in hundreds of pounds are the same as the class number. The wheel spacings are the same for all classes. The stresses for Cooper's loadings calculated for one class may be used to obtain the stresses due to any other class loading. For example, the live load stresses in any truss due to Cooper's Class E 60 are equal to $\frac{3}{2}$ of the stresses in the same truss due to Class E 40 loading. The E 50, E 55 and E 60 loadings are those most used for steam railways in the United States. In bridges designed for Class E 40 loading and under the floor system must in addition be designed for two moving loads of 50,000 lb. each, spaced 6 ft. apart on each track. The special loads for Class E 50 are 60,000 lb. with the same

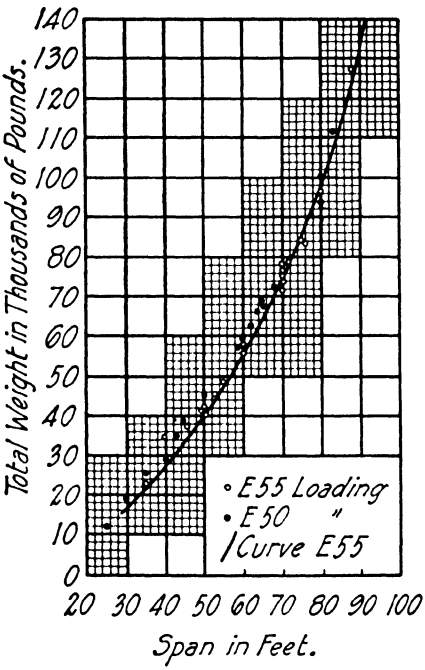


FIG. 7. WEIGHT OF SINGLE TRACK DECK PLATE GIRDER SPANS. OPEN TIMBER FLOOR. TYPE A4 (FLANGES OF 6 ANGLES WITHOUT COVER PLATES). CHICAGO, MILWAUKEE & ST. PAUL RY.

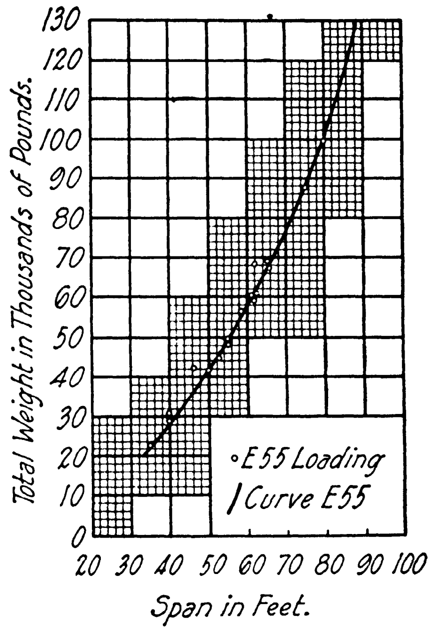


FIG. 8. WEIGHT OF SINGLE TRACK DECK PLATE GIRDER SPANS. TIMBER BALLAST FLOOR. TYPE A4 (FLANGES OF 6 ANGLES WITHOUT COVER PLATES). CHICAGO, MILWAUKEE & ST. PAUL RY.

spacing. The American Railway Engineering Association has adopted Cooper's loadings, except that the special loads are spaced 7 ft. The live loads used by several prominent railroads are given in Table XVI. The heaviest locomotives in use on American railroads as given in Bulletin No. 161, November 1913, of the Am. Ry. Eng.Assoc., by Mr. J. E. Greiner, Consulting Engineer, are given in Table II. The maximum stresses in terms of the maximum stresses for E 50 loading for spans between 100 ft. and 10 ft. are given in the last two columns. The ratios for spans greater than 100 ft. are less than for those given. The larger ratio is for short spans so that by increasing the special concentrated loads a bridge designed for an E 50 loading will safely carry the heaviest engines now in use.

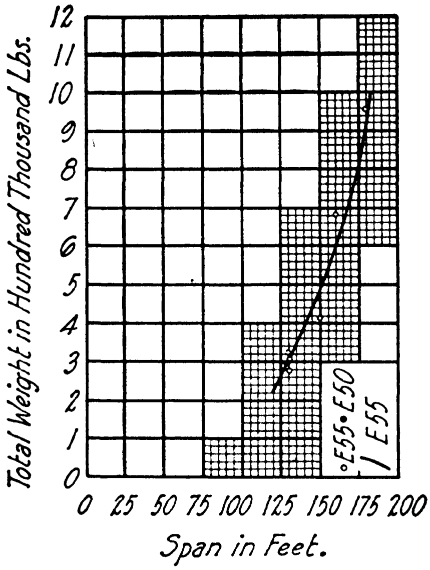


FIG. 9. WEIGHT OF SINGLE TRACK THROUGH RIVETED TRUSS SPANS. CHICAGO, MILWAUKEE & ST. PAUL RY.

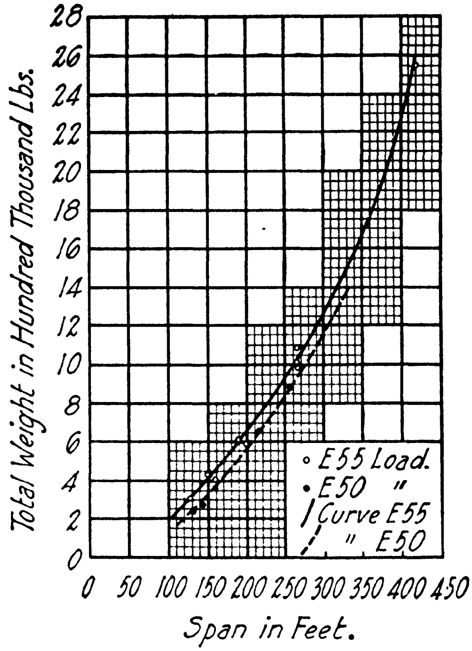


FIG. 10. WEIGHT OF SINGLE TRACK THROUGH PIN CONNECTED TRUSS SPANS. CHICAGO, MILWAUKEE & ST. PAUL RY.

TABLE II.§

HEAVIEST LOCOMOTIVES AND RELATIVE STRESSES PRODUCED FOR SPANS OF 10 FT. TO 100 FT.

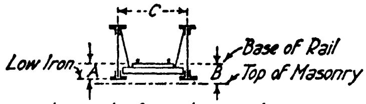
Class.	Engine Alone.			Double Header.*			Proportional Stress.	
	Weight in 1,000 Lb.	Wheel Base, Ft.	Proportional Weight.	Weight in 1,000 Lb.	Wheel Base, Ft.	Weight per Ft., Lb.	From	To
E 50†	225.0	23.00	1.00	710.0	104.0	6,830	1.00	1.00
Atlantic	214.8	30.79	.96	728.4	127.76	5,700	0.83	1.15
Prairie	244.7	34.25	1.09	807.5	132.92	6,070	0.88	1.03
Consolidation	260.1	26.50	1.16	860.4	131.81	6,520	0.99	1.14
12 Wheel	262.0	27.08	1.17	817.4	130.15	6,280	1.00	1.14
Decapod	267.0	29 83	1.19	802.0	127.00	6,320	0.96	1.07
Pacific	270.0	35.20	1.20	865.4	142.48	6,070	0.93	1.08
Mikado	305.0	35.00	1.36	960.0	150.00	6,400	1.02	1.16
12 Wheel Articulated‡	334.5	30.66	1.49	473.8	64.56	7,340	0.98	1.15
10 Coupled	361.0	43.50	1.60	1,074.0	161.00	6,670	1.00	1.26
20 Wheel Articulated‡	478.0	59.80	2.12	703.6	99.70	7,060	1.01	1.14
16 Wheel Articulated‡	493.0	40.17	2.19	588.0	82.58	7,130	1.26	1.34
24 Wheel Articulated‡	616.0	65.92	2.74	841.6	105.82	7,950	1.15	1.33
12 Wheel Electric Motor	300.4	38 50	1.33	600.8	86.50	6,950	0.83	0.98
16 Wheel Electric Motor	320.0	44.22	1.42	640.0	102.84	6,220	0.84	0.93

* Weight and wheel base for articulated engines are given for one engine and tender.

† Given for comparison.

‡ Mallet Type.

§ Also see Table XVII.



Shear in thousands of pounds per rail.
Loading – 2-188.75 ton engines followed by 6,000 lbs per foot uniform load.

Span	Total End Shear	A	B	C	Weight of Span
30'0"	98.0	2' 1 13/16"	2' 1 15/16"	15'0"	40 000 lbs.
35'0"	108.0	2' 2 1/16"	2' 1 13/16"	16'0"	48 000 "
40'0"	118.0	2' 2 1/16"	2' 1 15/16"	17'0"	58 000 "
45'0"	129.0	2' 2 5/16"	2' 2 1/16"	17'6"	68 000 "
50'0"	139.0	2' 2 7/16"	2' 2 1/16"	17'6"	77 000 "
55'0"	148.0	2' 2 11/16"	2' 9 1/8"	17'6"	88 000 "
60'0"	158.0	2' 2 7/8"	2' 9 1/8"	17'6"	98 000 "
65'0"	170.0	2' 3"	2' 9 1/8"	17'6"	111 000 "
70'0"	182.0	2' 3 3/16"	3' 9 1/8"	17'6"	120 000 "
75'0"	194.0	2' 3 5/16"	3' 10 1/4"	17'6"	133 000 "
80'0"	207.0	2' 2 7/8"	3' 10 1/4"	17'6"	149 000 lbs.
85'0"	220.0	2' 3 1/16"	3' 10 1/4"	17'6"	163 000 "
90'0"	233.0	2' 3 5/16"	3' 10 3/8"	17'6"	180 000 "
95'0"	246.0	2' 3 9/16"	3' 10 3/8"	17'6"	200 000 "
100'0"	260.0	2' 3 7/8"	3' 10 1/2"	17'6"	222 000 "
110'0"	280.0	2' 4 1/2"	3' 10 1/2"	17'6"	250 000 "

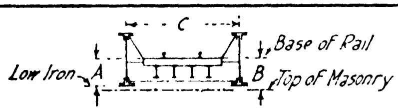
Span	Weight of one Light Girder	Weight of one Heavy Girder	Weight of one Floor
30'0" to 50'0"	0.22 W	0.39 W	0.56 W
55'0" to 80'0"	0.27 W	0.48 W	0.47 W
85'0" to 110'0"	0.31 W	0.57 W	0.38 W

I-Beams, 18" @ 65 lbs.

ERECTOR'S NOTE:–
W – Total weight of one single track span with two light girders.

DATA ON THROUGH PLATE GIRDER SPANS
I-BEAM FLOORS

FIG. 11. WEIGHTS OF THROUGH PLATE GIRDER SPANS.
ILLINOIS CENTRAL RAILROAD.



Shear in thousands of pounds per rail.
Loading – 2-188.75 ton engines, followed by 6000 lbs. per foot uniform load

Span	Total End Shear	A	B	C	Weight of Span
30'0"	100.5	3' 2 7/8"	3' 3 5/16"	15'6"	45 000 lbs
35'0"	111.9	3' 3 3/16"	3' 3 5/16"	16'6"	56 000 "
40'0"	122.5	3' 3 5/16"	3' 3 6/16"	17'6"	64 400 "
45'0"	132.6	3' 3 9/16"	3' 3 7/16"	17'6"	71 000 "
50'0"	142.8	3' 3 5/8"	3' 3 7/16"	17'6"	81 200 "
55'0"	153.4	3' 4 1/8"	3' 10 3/4"	17'6"	95 900 "
60'0"	161.1	3' 4 1/8"	3' 10 3/4"	17'6"	103 800 "
65'0"	174.9	3' 4 1/4"	3' 10 15/16"	17'6"	116 000 "
70'0"	187.4	3' 4 9/16"	3' 10 15/16"	17'6"	128 000 "
75'0"	201.9	3' 4 3/16"	4' 11 9/16"	17'6"	145 700 "
80'0"	213.4	3' 4 1/4"	4' 11 9/16"	17'6"	154 200 lbs
85'0"	228.1	3' 4 7/16"	4' 11 5/8"	17'6"	176 000 "
90'0"	240.6	3' 4 11/16"	4' 11 5/8"	17'6"	189 600 "
95'0"	254.7	3' 4 15/16"	4' 11 3/4"	17'6"	210 000 "
100'0"	267.2	3' 5 3/16"	4' 11 3/4"	17'6"	224 800 "
110'0"	293.6	3' 5 13/16"	4' 11 3/4"	17'6"	263 000 "

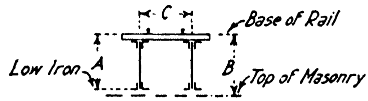
Span	Weight of one Light Girder	Weight of one Heavy Girder	Weight of one Floor
30'0" to 50'0"	0.24 W	0.42 W	0.54 W
55'0" to 80'0"	0.25 W	0.46 W	0.50 W
85'0 to 110'0"	0.28 W	0.51 W	0.43 W

ERECTOR'S NOTE:–
W – Total weight of one single track span with two light girders.

DATA ON
THROUGH PLATE GIRDER SPANS
STRINGER FLOOR

FIG. 12. WEIGHTS OF THROUGH PLATE GIRDER SPANS.
ILLINOIS CENTRAL RAILROAD.

Span	Total End Shear	A	B	C	Weight of Span
30'0"	94.0	4'10 7/8"	4'10 9/16"	7'0"	18 000 lbs
35'0"	103.5	5'2 3/8"	5'2 5/8"	7'0"	22 000 "
40'0"	113.5	5'9 1/2"	5'9 1/2"	7'0"	28 000 "
45'0"	123.5	6'3 1/2"	6'3 1/4"	7'0"	34 000 "
50'0"	132.5	6'9 3/4"	6'8 3/4"	7'0"	40 000 "
55'0"	141.0	7'1"	7'1 1/4"	7'0"	46 000 "
60'0"	150.0	7'4"	7'10 1/2"	7'0"	57 000 "
65'0"	165.0	8'6"	9'0 1/4"	8'0"	62 000 "
70'0"	176.0	9'4"	9'10 1/4"	8'0"	68 000 "
75'0"	189.0	9'6 1/4"	10'0 1/4"	8'0"	78 000 "
80'0"	202.0	9'8 7/8"	11'4 1/4"	8'0"	90 000 "
85'0"	216.0	10'0"	11'7 1/4"	9'0"	100 000 "
90'0"	228.0	10'2 3/16"	11'9 1/4"	9'0"	114 000 "
95'0"	242.0	10'2 3/8"	11'9 1/2"	9'0"	130 000 "
100'0"	259.0	10'4 5/8"	11'11 1/4"	9'0"	150 000 "
110'0"	295.0	10'9 3/8"	12'3 1/4"	9'6"	225 000 "



Shear in thousands of pounds per rail.
Loading – 2-188.75 ton engines, followed by 6000 lbs per foot uniform load

ERECTION NOTE:–
In all spans, 30'0" to 60'0" in length, one girder will weigh 45% of total weight of span
In all spans 65'0" to 110'0 in length, one girder will weigh 46.5 per cent of total weight of span.

DATA ON
DECK PLATE GIRDER
SPANS

FIG. 13. WEIGHTS OF DECK PLATE GIRDER SPANS.
ILLINOIS CENTRAL RAILROAD.

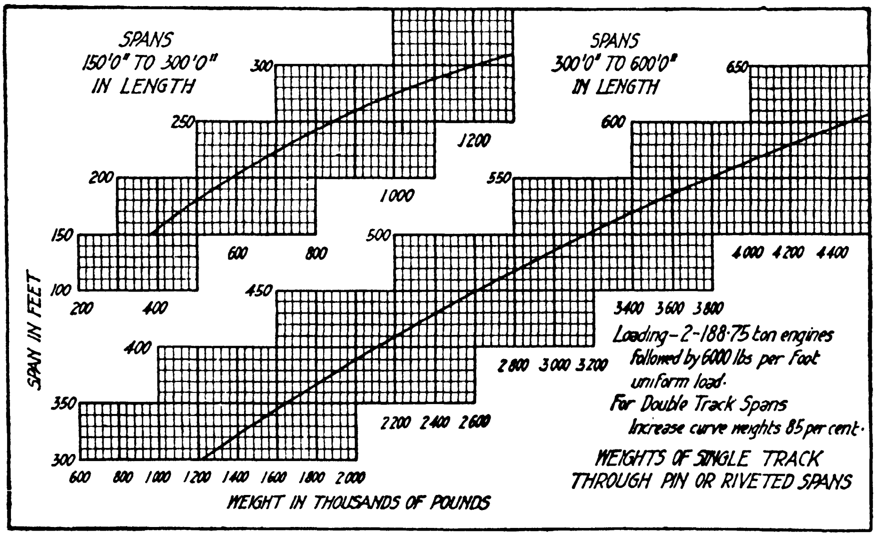


FIG. 14. WEIGHTS OF SINGLE TRACK THROUGH SPANS
ILLINOIS CENTRAL RAILROAD.

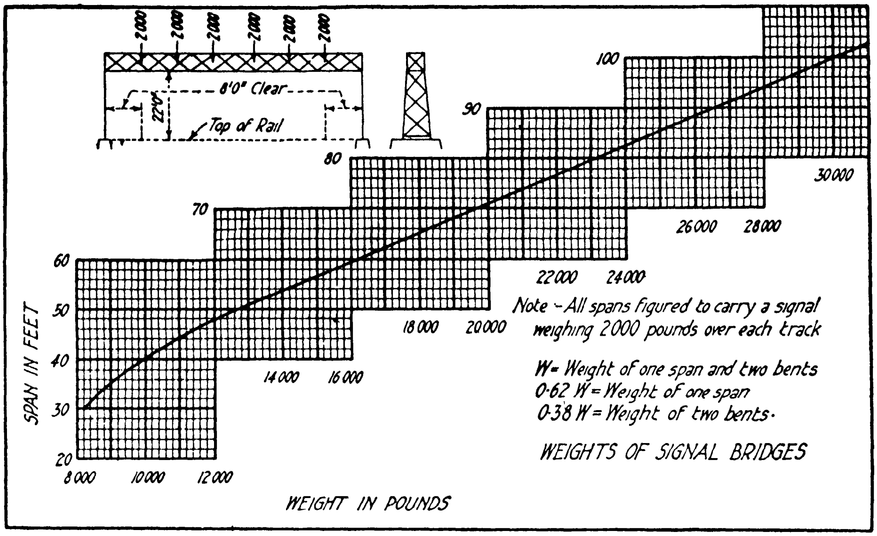


FIG. 15. WEIGHTS OF SIGNAL BRIDGES.
ILLINOIS CENTRAL RAILROAD.

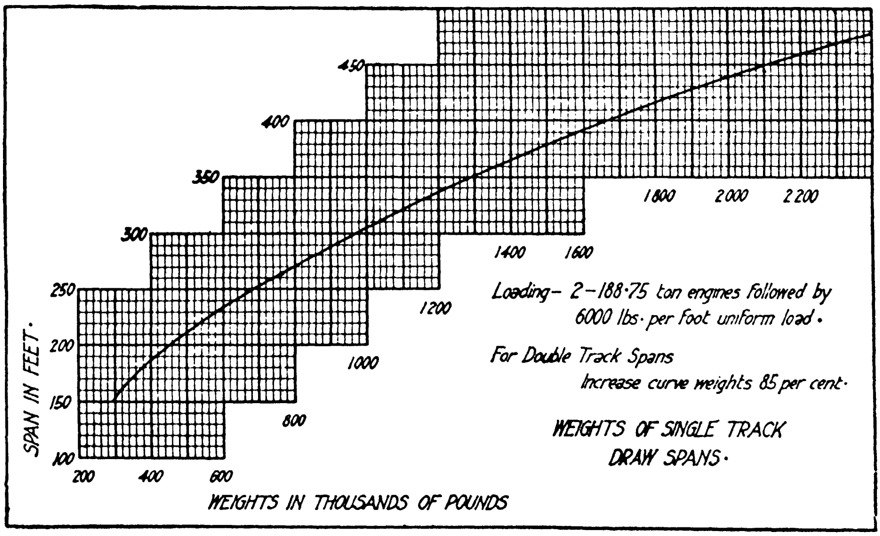


FIG. 16. WEIGHTS OF SINGLE TRACK DRAW SPANS.
ILLINOIS CENTRAL RAILROAD.

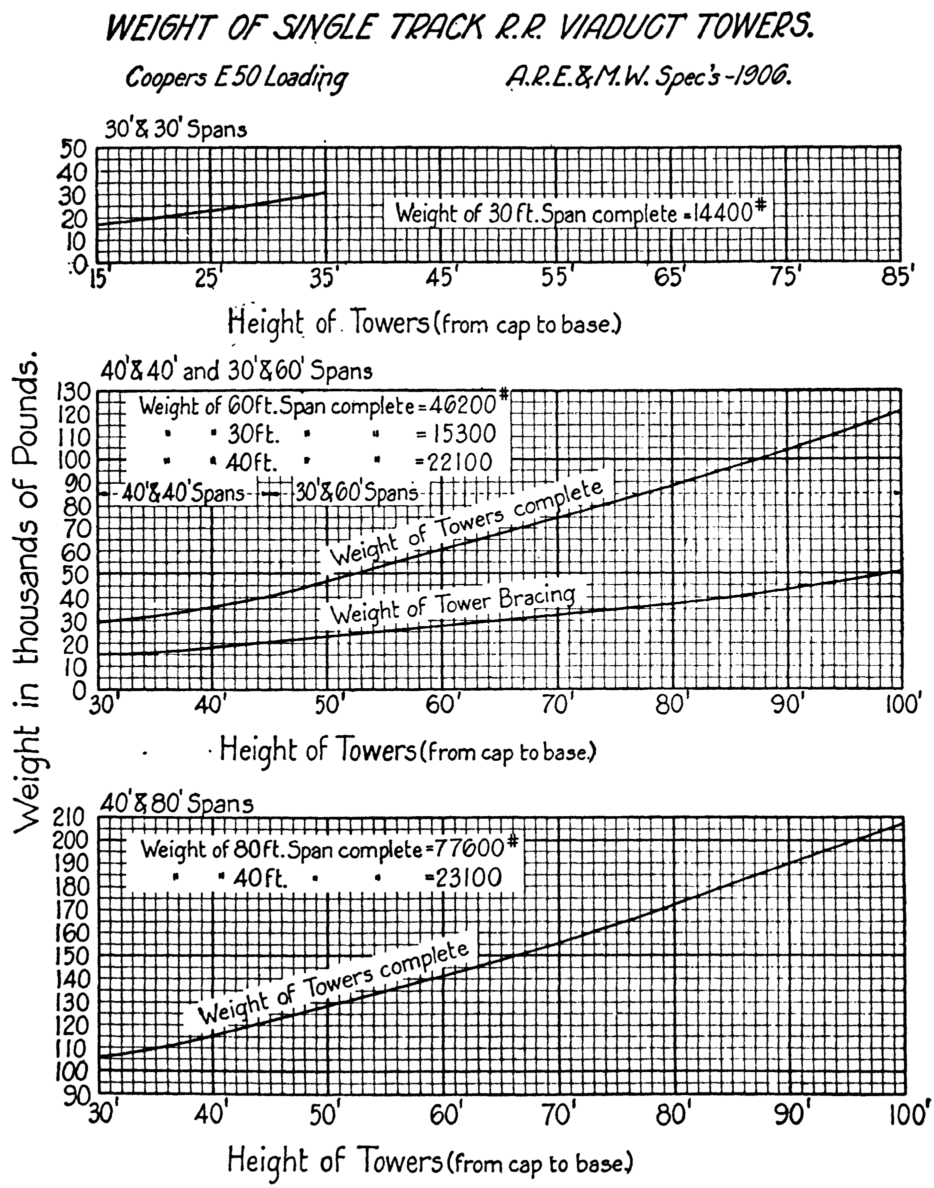


FIG. 17. WEIGHT OF STEEL VIADUCTS. MCCLINTIC-MARSHALL CONSTRUCTION CO.

Class	8'	5'	5'	5'	9'	5'	6'	5'	8'	8'	5'	5'	5'	9'	5'	6'	5'	5'	Uniform Load
E·40	20 000	40 000	40 000	40 000	40 000	26 000	26 000	26 000	26 000	20 000	40 000	40 000	40 000	40 000	26 000	26 000	26 000	26 000	4 000 lb. per lin. Ft.
E·45	22 500	45 000	45 000	45 000	45 000	29 250	29 250	29 250	29 250	22 500	45 000	45 000	45 000	45 000	29 250	29 250	29 250	29 250	4500 lb. per lin. Ft.
E·50	25 000	50 000	50 000	50 000	50 000	32 500	32 500	32 500	32 500	25 000	50 000	50 000	50 000	50 000	32 500	32 500	32 500	32 500	5 000 lb. per lin. Ft.
E·55	27 500	55 000	55 000	55 000	55 000	35 750	35 750	35 750	35 750	27 500	55 000	55 000	55 000	55 000	35 750	35 750	35 750	35 750	5 500 lb. per lin. Ft.
E·60	30 000	60 000	60 000	60 000	60 000	39 000	39 000	39 000	39 000	30 000	60 000	60 000	60 000	60 000	39 000	39 000	39 000	39 000	6 000 lb. per lin. Ft.

FIG. 18. COOPER'S CONVENTIONAL ENGINE LOADINGS.
(Loads for one track.)

Equivalent Uniform Load System.—The equivalent uniform load for calculating the stresses in trusses and the bending moments in beams, is the uniform load that will produce the same bending moment at the quarter points of the truss or beam as the maximum bending moment produced by the wheel concentrations. The equivalent uniform loadings for different spans for Cooper's E 40 loading are given in Fig. 19. The equivalent uniform loading for E 60 loading will be $\frac{3}{2}$ the values for E 40 in Fig. 19. In calculating the stresses in the truss members select

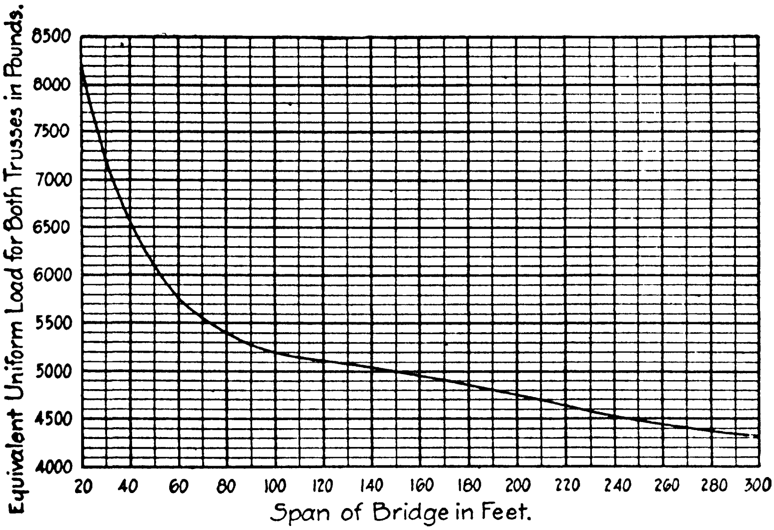


FIG. 19. EQUIVALENT UNIFORM LIVE LOAD FOR COOPER'S E40 LOADING.
(Loads for one track.)

the equivalent load for the given span, and calculate the chord and web stresses by the use of equal joint loads, as for highway bridges. In designing the stringers for bending moment take a loading for a span equal to one panel length, and for the maximum floorbeam reaction take a

loading for a span equal to two panel lengths. It is necessary to calculate the maximum end shears and the shears at intermediate points by wheel concentrations, or to use equivalent uniform loads calculated for wheel concentrations. The calculated values of the moment, M, shear, S, and floorbeam reaction, R, for Class E 60 are given in Table III. The equivalent uniform load method has been advocated very strongly by Mr. J. A. L. Waddell who has described its use in detail in his "De Pontibus." Live load stresses as calculated by the method of equivalent uniform loads are too small for the chords and webs between the ends of the truss and the quarter points, and are too large between the quarter points. The stresses obtained for the counters are too large. The live load stresses calculated by the method of equivalent uniform loads are sufficiently accurate for all practical purposes. Even though the equivalent uniform load method is simple to apply and gives results which are sufficiently accurate, it is now seldom used.

Uniform Load and One or Two Excess Loads.—A uniform load is used and to provide for the wheel concentrations one or two excess loads are assumed to run on top of the uniform load. This method is now rarely used. In a paper entitled "Rolling Loads on Bridges," published in Bulletin No. 161, Am. Ry. Eng. Assoc., November 1913, Mr. J. E. Greiner, Consulting Engineer, found that thirty-eight of the thirty-nine most important railroads in the country used a system of wheel concentrations, and one road used a uniform load with a single excess load; the method of equivalent uniform loads was not used.

MAXIMUM STRESSES.—The conditions of live loading for maximum stresses in beams and trusses are as follows.

Uniform Live Load on Beam or Girder.—For bending moment the span should be fully loaded. For shear the longer segment of the span should be loaded.

Equal Joint Loads.—For bending moment (chord stresses) the bridge should be fully loaded. For shear (web stresses in trusses with parallel chords) the longer segment of the truss should be loaded for maximum stress, and the shorter segment of the truss should be loaded for maximum counter stress (minimum stress).

Point of Maximum Bending Moment in a Beam.—The maximum bending moment in a beam loaded with moving loads will come under a heavy load when this load is as far from one end of the beam as the center of gravity of all the moving loads then on the beam is from the other end of the beam.

Wheel Loads, Bridge with Parallel Chords.—The maximum bending moment at any joint in the loaded chord will occur when the average load on the left of the section is equal to the average load on the entire span.

The maximum bending moment at any joint in the unloaded chord of a symmetrical Warren truss will occur when the average load on the entire span is equal to the average load on the left of the section, one-half of the load on the panel under the joint being considered as part of the load on the left of the section.

The maximum shear in any panel of a truss will occur when the average load on the panel is equal to the average load on the entire bridge.

Wheel Loads, Bridge with Inclined Chords.—The criterion for maximum bending moment in a bridge with vertical posts is the same as for bridges with parallel chords.

For web members the criterion is that

$$P/L = P_2(1 + a/e)/l \tag{1}$$

where P = total load on the bridge;
P_2 = load on the panel in question;
L = span of bridge;
l = panel length;
a = distance from left abutment to left end of panel in question;
e = distance from left abutment to intersection of top chord section of the panel produced and the lower chord. (The intersection is to the left and outside of the span.)

KINDS OF STRESS.—Bridges must be designed for the stresses due to (1) dead load; (2) live or moving load; (3) wind load; (4) snow load; (5) impact stresses; (6) temperature stresses; (7) centrifugal stresses, and (8) secondary stresses not taken into account in the calculations. In addition to the above it is necessary in determining the allowable stress in any member to take into account imperfections in materials and workmanship, possible increase in live loads, fatigue of metals, the frequency of the application of the stress, corrosion and deterioration of materials, etc. The structure should be so designed that no part will be ever stressed beyond the elastic limit. The allowable stresses for dead load are usually taken at about 60 to 70 per cent of the elastic limit; for an elastic limit of 30,000 lb., the allowable working stresses for dead loads alone would then vary from 18,000 to 21,000 lb. per sq. in.

IMPACT STRESSES.*—As a load moves over the bridge it causes shocks and vibrations whereby the actual stresses are increased over those due to the static load alone. It is shown in mechanics of materials that a load suddenly applied to a bar or beam will produce stresses twice the stresses produced by the same load gradually applied. A bridge is a complex structure and it is not possible to determine the exact effect of the moving loads. It has been found by experiment that the ultimate strength for repeated loads is much less than for dead loads. In a bridge it will be seen that the dead load is a fixed load and that the live load is a varying load.

For stresses of one kind Professor Launhardt has proposed the following formula:

$$P = S\left(1 + \frac{\text{Min. stress}}{\text{Max. stress}}\right) \tag{2}$$

where P is the allowable working stress required, and S is the allowable working stress for live loads, varying from zero to the maximum stress. For stresses of opposite kinds Professor Weyrauch has proposed the following formula:

$$P = S\left(1 - \frac{\text{Min. stress}}{2\ \text{Max. stress}}\right) \tag{3}$$

where P and S are the same as for the Launhardt formula, the maximum and minimum stresses being taken without sign. For columns and struts the allowable stresses as given by formulas (2) and (3) are to be reduced by a suitable column formula.

There are three methods in common use for taking account of impact and fatigue: (1) Impact formulas; (2) Launhardt-Weyrauch formulas, and (3) Cooper's Method.

(1) **Impact Formulas.**—The formula in most common use is given in the form

$$I = S\left(\frac{a}{L + b}\right) \tag{4}$$

where I = impact stress to be added to the static live load stress, S = the static live load stress, L = the length in feet of the portion of the bridge that is loaded to produce the maximum stress in the member, and a and b are constants expressed in feet. The American Railway Engineering Association specifies for railway bridges, $a = b = 300$ ft. Mr. J. A. L. Waddell specifies $a = 400$ ft., and $b = 500$ ft. for railway bridges; and $a = 100$ ft., and $b = 150$ ft. for highway bridges. For the names of several roads using A. R. E. A. impact formula, see Table XVI.

For highway bridges the American Bridge Company specifies that the maximum live load stress shall be increased 25 per cent to cover impact and vibration.

Mr. C. C. Schneider, M. Am. Soc. C. E., specifies that for electric railway bridges

$$I = S \cdot 150/(L + 300) \tag{5}$$

In the Osborn Engineering Company's 1901 specifications for railway and for highway bridges the impact is calculated by the formula

$$I = S \cdot S/(S + D) \tag{6}$$

* Also see discussion of 1924 specifications and A. R. E. A. 1920 specifications in last part of this chapter.

where S is the static live load stress and D is the dead load stress. This method is used by the Illinois Central R. R.

(2) **Launhardt-Weyrauch Formulas.**—Formula (2) is used for determining the allowable stress for stresses of one kind and formula (3) is used for determining the allowable stress for stresses of different kinds. This method is used in Thatcher's Specifications, in Common Standard Specifications (Harriman Lines), and specifications of Pennsylvania Lines West of Pittsburgh.

(3) **Cooper's Method.**—Cooper uses formula (2) and calculates the area for the dead load and the area for the live load stress separately. For dead loads from formula (2) we have $P = 2S$, while for live loads the range of stress is from zero to the maximum, and $P = S$.

For a reversal of stress Cooper designs the member to take both kinds of stress, but to each stress he adds eight-tenths of the lesser of the two stresses.

IMPACT TESTS.—The American Railway Engineering Association has made an exhaustive series of tests to determine the effect of impact on railway bridges. The following summary is taken from the Proceedings of Am. Ry. Eng. Assoc., Vol. 12, Part 3.

(1) With track in good condition the chief cause of impact was found to be the unbalanced drivers of the locomotive. Such inequalities of track as existed on the structures tested were of little influence on impact on girder flanges and main truss members of spans exceeding 60 to 75 ft. in length.

(2) When the rate of rotation of the locomotive drivers corresponds to the rate of vibration of the loaded structure, cumulative vibration is caused, which is the principal factor in producing impact in long spans. The speed of the train which produces this cumulative vibration is called the "critical speed." A speed in excess of the critical speed, as well as a speed below the critical speed, will cause vibrations of less amplitude than those caused at or near the critical speed.

(3) The longer the span length the slower is the critical speed and therefore the maximum impact on long spans will occur at slower speeds than on short spans.

(4) For short spans, such that the critical speed is not reached by the moving train, the impact percentage tends to be constant so far as the effect of counterbalance is concerned, but the effect of rough track and wheels becomes of greater importance for such spans.

(5) The impact as determined by extensometer measurements on flanges and chord members of trusses is somewhat greater than the percentages determined from measurements of deflection, but both values follow the same general law.

(6) The maximum impact on web members (excepting hip verticals) occurs under the same conditions which cause maximum impact on chord members, and the percentages of impact for the two classes of members are practically the same.

(7) The impact on stringers is about the same as on plate girder spans of the same length and the impact on floorbeams and hip verticals is about the same as on plate girders of a span equal to two panels.

(8) The maximum impact percentage as determined by these tests is closely given by the formula

$$I = \frac{100}{1 + \frac{l^2}{20,000}} \tag{7}$$

in which I = impact percentage and l = span length in feet.

(9) The effect of differences of design was most noticeable with respect to differences in the bridge floors. An elastic floor, such as furnished by long ties supported on widely spaced stringers, or a ballasted floor, gave smoother curves than were obtained with more rigid floors. The results clearly indicated a cushioning effect with respect to impact due to open joints, rough wheels and similar causes. This cushioning effect was noticed on stringers, hip verticals and short span girders.

(10) The effect of design upon impact percentage for main truss members was not sufficiently marked to enable conclusions to be drawn. The impact percentage here considered refers to variations in the axial stresses in the members, and does not relate to vibrations of members themselves.

(11) The impact due to the rapid application of a load, assuming smooth track and balanced loads, is found to be from both theoretical and experimental grounds, of no practical importance.

(12) The impact caused by balanced compound and electric locomotives was very small and the vibrations caused under the loads were not cumulative.

(13) The effect of rough and flat wheels was distinctly noticeable on floorbeams, but not on truss members. Large impact was, however, caused in several cases by heavily loaded freight cars moving at high speeds.

TABLE III.

Maximum Moments, M; End Shears, S; and Floorbeam Reactions, R; Per Rail, for Girders.

Cooper's E60 Loading (A. R. E. A.).

Loading Two E 60 Engines and Train Load of 6,000 Pounds per Foot or Special Loading Two 75,000 Pound Axle Loads 7 Ft. C. to C.

Moments in Thousands of Foot-Pounds. Shears and Floorbeam Reactions in Thousands of Pounds.

Results for One Rail. Results from Special Loading marked*. A. R. E. A. Impact Formula.

Span L, Ft.	Maximum Moments M.	Moment Impact M′.	End Shear S.	End Shear Impact S′.	Floorbeam Reaction R.	Floorbeam Impact R′.	Span L, Ft.	Maximum Moments M.	Moment Impact M′.	End Shear S.	End Shear Impact S′.
5	* 46.9	* 46.1	*37.5	*36.9	*37.5	*36.3	50	1426.3	1222.6	130.8	112.1
6	* 56.2	* 55.1	*37.5	*36.8	40.0	38.5	51	1474.7	1260.4	132.5	113.2
7	* 65.6	* 64.2	38.6	37.7	47.1	45.0	52	1522.8	1297.8	134.1	114.3
8	* 75.0	* 73.0	*42.2	*41.2	52.5	49.8	53	1571.0	1335.1	135.7	115.3
9	* 84.4	* 82.0	*45.8	*44.5	56.7	53.5	54	1621.5	1374.2	137.4	116.4
10	* 93.7	* 90.7	*48.8	*47.2	60.0	56.3	55	1675.2	1415.7	139.0	117.5
11	*103.0	* 99.5	*51.1	*49.3	65.5	61.0	56	1728.0	1456.7	140.6	118.5
12	120.0	115.4	*53.2	*51.1	70.0	64.8	57	1781.9	1497.4	142.2	119.5
13	142.5	136.6	55.4	53.1	73.9	68.0	58	1834.5	1537.4	143.8	120.5
14	165.0	157.6	57.8	55.2	78.2	71.5	59	1891.4	1580.6	145.4	121.5
15	187.5	178.6	60.0	57.2	82.0	74.5	60	1949.4	1624.5	147.0	122.5
16	210.0	199.3	63.8	60.6	85.3	77.1	61	2007.5	1668.3	148.6	123.5
17	232.5	220.0	67.1	63.5	88.2	79.2	62	2064.3	1710.8	150.2	124.5
18	255.0	240.5	70.0	66.0	91.0	81.3	63	2123.4	1754.9	152.0	125.6
19	280.0	263.2	72.6	68.3	94.3	83.7	64	2183.3	1799.4	153.8	126.8
20	309.5	290.5	75.0	70.3	98.3	86.7	65	2246.3	1846.3	155.7	128.0
21	339.0	316.8	77.1	72.1	101.9	89.4	66	2309.3	1893.0	157.5	129.1
22	368.5	343.3	79.1	73.7	105.2	91.7	67	2378.3	1943.2	159.6	130.5
23	398.2	369.8	80.9	75.1	108.2	93.8	68	2435.4	1985.3	161.7	131.8
24	427.8	396.1	83.1	76.9	110.9	95.6	96	2498.4	2031.2	163.8	133.2
25	457.5	422.3	85.2	78.6	113.5	97.3	70	2561.3	2076.8	165.8	134.4
26	487.2	448.3	87.1	80.2	116.6	99.4	71	2624.5	2122.2	167.7	135.6
27	516.9	474.2	88.9	81.6	120.1	101.8	72	2688.0	2168.0	170.0	137.1
28	548.3	501.5	90.6	82.9	123.4	104.0	73	2750.9	2212.5	172.2	138.5
29	582.0	530.7	92.3	84.2	126.5	106.0	74	2818.5	2260.7	174.4	139.9
30	615.8	559.8	94.6	86.0	129.4	107.8	75	2888.6	2310.9	176.5	141.2
31	649.3	588.5	96.6	87.5	132.7	110.0	76	2958.0	2360.1	178.6	142.5
32	683.2	617.3	98.6	89.1	136.5	112.5	77	3028.6	2410.0	180.6	143.7
33	716.9	645.8	100.4	90.5	140.0	114.8	78	3096.6	2457.6	182.5	144.8
34	750.6	674.2	102.1	91.7	143.2	116.7	79	3168.2	2507.8	184.4	146.0
35	784.5	702.5	103.8	93.0	146.4	118.7	80	3240.7	2558.5	186.3	147.1
36	823.0	734.9	105.9	94.6	149.3	120.4	81	3311.4	2607.4	188.4	148.4
37	861.6	767.0	107.8	96.0	152.2	122.1	82	3385.1	2658.4	190.4	149.5
38	900.0	798.8	109.7	97.4	155.6	124.2	83	3459.6	2709.8	192.3	150.6
39	940.0	831.8	111.4	98.6	158.8	126.0	84	3534.6	2761.4	194.2	151.7
40	983.4	867.7	113.1	99.8	162.0	127.9	85	3610.4	2813.3	196.1	152.8
41	1027.0	903.5	115.2	101.3	....	....	86	3689.4	2867.4	198.1	154.0
42	1070.4	938.9	117.2	102.8	....	....	87	3766.5	2919.8	200.1	155.1
43	1113.9	974.2	119.0	104.1	....	....	88	3846.0	2973.7	202.1	156.3
44	1157.4	1009.4	120.8	105.3	....	....	89	3924.3	3026.5	204.0	157.3
45	1201.1	1044.4	122.5	106.5	Viaduct	....	90	4005.8	3081.4	205.8	158.3
46	1244.4	1078.9	124.2	107.7	Span	....	91	4084.4	3133.8	207.7	159.4
47	1287.9	1113.4	125.9	108.8	30′–60′	....	92	4164.0	3186.7	209.7	160.5
48	1331.4	1147.8	127.5	109.9	179.2	....	93	4246.6	3241.6	211.6	161.5
49	1378.3	1184.8	129.2	111.1	....	....	94	4328.0	3295.4	213.5	162.6

Note. Impact in this table is 1910, A. R. E. A.

TABLE III.—*Continued.*

MAXIMUM MOMENTS, M; END SHEARS, S; AND FLOORBEAM REACTIONS, R; PER RAIL, FOR GIRDERS.

Cooper's E60 Loading (A. R. E. A.).

Span L, Ft.	Maximum Moments M.	Moment Impact, M'.	End Shear S.	End Shear Impact S'.	Floorbeam Reaction R.	Floorbeam Impact R'.	Span L, Ft.	Maximum Moments M.	Moment Impact M'.	End Shear S.	End Shear Impact S'.
95	4408.4	3348.2	215.4	163.6	Viaduct	....	110	5829.6	4265.5	243.0	177.8
96	4490.7	3402.0	217.2	164.5	Span	....	111	5937.4	4333.9	244.8	178.7
97	4573.5	3456.0	219.2	165.6	40'-60'	....	112	6040.0	4398.1	246.6	179.5
98	4659.8	3512.4	221.2	166.7	197.2	....	113	6148.2	4466.0	248.3	180.3
99	4743.8	3566.7	223.1	167.7	....	....	114	6258.0	4534.8	250.0	181.2
100	4830.0	3622.5	225.0	168.8	Viaduct	....	115	6366.8	4602.5	251.8	182.0
101	4916.9	3678.5	226.8	169.7	Span	....	116	6478.0	4671.6	253.6	182.9
102	5004.0	3734.4	228.6	170.6	40'-80'	....	117	6586.1	4738.2	255.3	183.6
103	5115.5	3808.1	230.4	171.5	236.5	....	118	6696.6	4806.1	257.0	184.4
104	5212.8	3870.9	232.3	172.5	....	....	119	6808.3	4874.7	258.8	185.3
105	5306.5	3930.7	234.1	173.4	....	....	120	6921.6	4944.0	260.5	186.1
106	5401.3	3991.1	235.9	174.3	....	....	121	7030.5	5009.9	262.2	186.9
107	5499.2	4053.4	237.7	175.2	....	....	122	7143.8	5078.5	264.0	187.7
108	5617.0	4130.1	239.4	176.0	....	....	123	7260.1	5148.9	265.7	188.4
109	5727.6	4201.1	241.2	176.9	....	....	124	7376.4	5219.1	267.4	189.2
							125	7495.2	5290.7	269.1	190.0

CALCULATION OF STRESSES.—For the calculation of stresses in railway bridges, see the author's "The Design of Highway Bridges;" Johnson, Bryan & Turneaure's "Framed Structures," Part I; Marburg's "Framed Structures," Part I; Spofford's "Theory of Structures"; or other standard textbook.

Moments, End Shears and Floorbeam Reactions.—The maximum bending moments and end shears, for Cooper's E 60, and A. R. E. A. special loadings, for girders up to 125 ft. span are given in Table III. The maximum moments occur at a point near the center of the girder. Maximum floorbeam reactions are given for stringers up to 40 ft. span. The table also gives the impact stress calculated for A. R. E. A. impact formula (4).

The maximum moments, end shears, quarter-point shears, center shears, and maximum floorbeam reactions for girders up to 75 ft. span are given in Table IV.

Moment Diagram.—A diagram giving the position of the wheels in Cooper's E loadings that will produce maximum moment in a beam or at a panel point in a truss is given in Table Va. The condition for maximum shear in the first panel is the same as for bending moment at L_1, which value may be obtained from Table Va. Other loadings for maximum shear must be calculated by means of the criterion given above.

A moment diagram for Cooper's E 60 loading is given in Table Vb, and brief instructions for use of the table are given on the page opposite Table Vb.

Shears in Bridges.—Shears in the panels of the loaded chords of spans with 3 to 9 panels, for Cooper's E 50 loading, are given in Table VI, Table VII, and Table VIII. To obtain the shears for E 60 loading multiply the tabular values by $\frac{6}{5}$. The stresses in the web members of a Pratt truss are equal to the shears $\times$ sec θ, where θ is the angle that each web member makes with a vertical line. The tables were calculated by the McClintic-Marshall Construction Company.

Moments in Bridges.—Bending Moments in beams and girders and at points in the loaded chord of bridges, are given in Table IX and Table X. The bending moments for an E 60 loading will be equal to the tabular values $\times \frac{6}{5}$.

For example, the bending moment for an E 50 loading, at joint L_1, in an 8 panel truss of 200-ft. span from Table X, is 6,787 thousand ft.-lb. For an E 60 loading the bending moment at joint L_1 is $6{,}787 \times 6/5 = 8{,}145$ thousand ft.-lb., which checks the value calculated from Table Vb on the page opposite Table Vb. The tables were calculated by the McClintic-Marshall Construction Company.

Elevated Trestle Span Reactions.—The floorbeam reactions and the maximum reactions of the intermediate and tower spans of elevated railway trestles may be calculated from Table IX and Table X, as follows:

Required the end reactions for a 40 ft. tower span and an 80 ft. intermediate span. Take a span equal to $40 + 80 = 120$ ft., and calculate the bending moment at a point 40 ft. from the left end. In Table IX, take a 6-panel bridge with 20 ft. panels, the bending moment at L_2 is

$M = 5{,}255$ thousand ft.-lb. Then the reaction, $R = M \times \dfrac{40 + 80}{40 \times 80} = M \times 3/80 = 5.255 \times 3/80 = 197.1$ thousand lb. For E 60, $R_1 = R \times 6/5 = 197.1 \times 6/5 = 236.5$ thousand lb., which checks the value in Table III.

TABLE IV.

Maximum End Shears, Quarter-Point Shears, Center Shears; Maximum Moments, and Floorbeam Reactions for Girders.

Cooper's E60 Loading (A. R. E. A.).

Moments in Thousands of Foot-Pounds. Shears and Floorbeam Reactions in Thousands of Pounds.

Results for One Rail. Results from Special Loading marked*.

Span L, Ft.	End Shear.	Quarter Point Shear.	Center Shear.	Maximum Moment.	Floorbeam Reaction.	Span L, Ft.	End Shear.	Quarter Point Shear.	Center Shear.	Maximum Moment.
10	*48.8	30.0	*18.8	* 93.7	60.0	45	122.5	75.3	35.2	1201.1
11	*51.1	*32.4	*18.8	*103.0	65.5	46	124.2	76.1	35.6	1244.4
12	*53.2	*34.4	*18.8	120.0	70.0	47	125.9	77.1	36.0	1287.9
13	55.4	*36.0	*18.8	142.5	73.9	48	127.5	78.2	36.3	1331.4
14	57.8	*37.5	19.3	165.0	78.2	49	129.2	79.2	36.8	1378.3
15	60.0	*38.8	*20.0	187.5	82.0	50	130.8	80.2	37.2	1426.3
16	63.8	*39.9	*21.1	210.0	85.3	51	132.5	81.2	37.8	1474.7
17	67.1	41.1	*22.1	232.5	88.2	52	134.1	82.2	38.3	1522.8
18	70.0	42.6	*22.9	255.0	91.0	53	135.7	83.1	38.7	1571.0
19	72.6	43.8	*23.7	280.0	94.3	54	137.4	84.1	39.2	1621.5
20	75.0	45.0	*24.4	309.5	98.3	55	139.0	85.2	39.6	1675.2
21	77.1	47.2	*25.0	339.0	101.9	56	140.6	86.3	40.0	1728.0
22	79.1	49.2	*25.6	368.5	105.2	57	142.2	87.3	40.4	1781.9
23	80.9	50.8	*26.1	398.2	108.2	58	143.8	88.3	40.8	1834.5
24	83.1	52.5	*26.6	427.8	110.9	59	145.4	89.3	41.3	1891.4
25	85.2	54.0	*27.0	457.5	113.5	60	147.0	90.2	41.8	1949.4
26	87.1	55.4	*27.4	487.2	116.6	61	148.6	91.1	42.3	2007.5
27	88.9	56.7	*27.8	516.9	120.1	62	150.2	92.0	42.8	2064.3
28	90.6	57.9	*28.1	548.3	123.4	63	152.0	92.9	43.2	2123.4
29	92.3	59.0	*28.5	582.0	126.5	64	153.8	93.8	43.7	2183.3
30	94.6	60.0	*28.8	615.8	129.4	65	155.7	94.7	44.1	2246.3
31	96.6	61.2	*29.1	649.3	132.7	66	157.5	95.6	44.6	2309.3
32	98.6	62.4	*29.3	683.2	136.5	67	159.6	96.5	45.0	2378.3
33	100.4	63.6	*29.6	716.9	140.0	68	161.7	97.4	45.4	2435.4
34	102.1	64.7	*29.8	750.6	143.2	69	163.8	98.3	45.7	2498.4
35	103.8	65.7	30.3	784.5	....	70	165.8	99.2	46.2	2561.3
36	105.9	66.7	30.9	823.0	....	71	167.7	100.1	46.6	2624.5
37	107.8	67.5	31.5	861.6	....	72	170.0	101.0	47.1	2688.0
38	109.7	68.3	32.0	900.0	....	73	172.2	101.9	47.5	2750.9
39	111.4	69.0	32.5	940.0	....	74	174.4	102.8	48.0	2818.5
40	113.1	70.2	33.0	983.4	....	75	176.5	103.6	48.4	2888.6
41	115.2	71.3	33.5	1027.0	....					
42	117.2	72.3	33.9	1070.4	....					
43	119.0	73.3	34.4	1113.9	....					
44	120.8	74.3	34.8	1157.4	....					

Position of Wheels for Maximum Moment; Table Va.

The wheel loads that will produce maximum moment at a point a given distance from the left end of a beam, or at any loaded panel point in a bridge, are given in Table Va. For example in an 8-panel Pratt truss of 200 ft. span, maximum moment at panel point L_1, 25 ft. from the left end, occurs with wheel No. 4, at the point; a maximum moment at L_2 occurs with wheel No. 7 at the point; etc.

Instructions for Use of Moment Table; Table Vb.

Line (1) is summation of loads from head of uniform load.
Line (2) is summation of loads from wheel No. 1.
Line (3) is the number of each wheel from wheel No. 1.
Line (4) is amount of each wheel load in thousand pounds.
Line (5) is distance c. to c. of the wheels, in feet.
Line (6) is distance of any wheel, or the head of uniform load, from wheel No. 1.

Line (7) is distance of any wheel from head of uniform load.

Line (8) is summation of moments of all wheels to right of any wheel, including the wheel in question, about head of uniform load.

Lines (9) to (25) are summations of moments of all wheels to left of the stepped line, including wheel on left of value, about the wheel just above the heavy vertical stepped line on each line.

The values to the right of the stepped lines are moments about the stepped line, including wheel to right of moment value given.

EXAMPLES.—Problem 1.—Calculate moment of wheels Nos. 1 to 15, inclusive, about wheel No. 15.

Follow vertical line passing through wheel No. 15 down to stepped line, and follow over to the left on line (12), and find 16,220 thousand ft.-lb. to right of vertical line through wheel No. 1.

Problem 2.—Calculate the moment of wheels Nos. 17, 16, 15, 14, about wheel No. 13.

Follow vertical line passing through wheel No. 13 down to the stepped line, and follow line (14) to right, and to left of the vertical line through wheel No. 17, find 1,281 thousand ft.-lb.

Problem 3.—Given a 200-ft. span, 8 panel Pratt railway bridge. The moments and shears are calculated as follows:

Moments.—Panel point L_1. From Table Va, there will be a maximum moment at L_1 with wheel No. 4 at the joint; and from Table Vb, line (7) it is 91 ft. from wheel No. 4 to the end of the uniform load, and it is also 175 ft. from joint L_1 to the end of the bridge, and there will be $175 - 91 = 84$ ft. of uniform load on the bridge. Then, $R_1 \times 200 = 24{,}550 + 426 \times 84 + 3 \times 84^2/2 = 70{,}918$ thousand ft.-lb.; and $R_1 = 354.6$ thousand lb. The moment at L_1 is $M_1 = 354.6 \times 25 - 720 = 8{,}145$ thousand ft.-lb.

Shear in Panel L_0L_1 is $S_1 = R_1 - 720/25 = 354.6 - 28.8 = 325.8$ thousand lb. (720 is the moment of wheels Nos. 1, 2, 3, about wheel No. 4).

TABLE Va.

Loading for Maximum Moment in Bridges for Cooper's Loadings.

Spans	10'	15'	20'	25'	30'	35'	40'	45'	50'	55'	60'	65'	70'	80'	90'	100'	110'	120'	130'	140'
300' to 260'	2	3	3	4	4	5	5	6	7	7	8	9	10	11	12	13	14	15	17	18
250' to 200'	2	3	3	4	4	5	5	6	7	8	8	9	10	11	12	13	14	15	17	18
190' to 150'	2	3	3	4	4	5	5	6	7	8	9	9	11	12	12	13	14	15	17	18
140'	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	13	14	15	17	18
130'	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	13	14	15	17	
120'	3	3	3	4	4	5	5	6	7	8	9	10	11	12	13	13	14	15		
110'	3	3	3	4	4	5	6	7	7	8	9	10	11	12	13	13	14			
100'	3	3	3	4	5	5	6	$\overline{14}$	$\overline{14}$	$\overline{14}$	$\overline{13}$	$\overline{13}$	11	12	13	13				
90'	3	3	4	4	5	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	12	13					
80'	3	3	4	4	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	12						
70'	3	3	4	4	$\overline{13}$	$\overline{13}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{11}$	$\overline{11}$	11							
65'	3	3	4	4	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{11}$	$\overline{11}$	11								
60'	3	3	4	4	5	$\overline{13}$	$\overline{12}$	$\overline{11}$	$\overline{11}$	$\overline{11}$	11									
55'	12	12	12	4	12	$\overline{13}$	$\overline{12}$	$\overline{12}$	$\overline{13}$	11										
50'	12	12	12	12	12	$\overline{13}$	$\overline{13}$	$\overline{13}$	12											
45'	12	12	12	12	12	$\overline{13}$	$\overline{13}$	13												
40'	3	3	3	12	12	$\overline{13}$	13													
35'	3	3	4	4	13	13														
30'	3	3	4	4	13															
25'	3	3	4	4																
20'	$\overline{4}$	3	4																	
15'	3	3																		
10'	3																			

WHEEL DETERMINING MAXIMUM MOMENT
COOPER'S LOADINGS
C·M·& ST·P·RY·

The shorter span is ahead followed by the longer one except wheel is over-lined·

TABLE V*b*.

MOMENT TABLE FOR COOPER'S E60 LOADING.

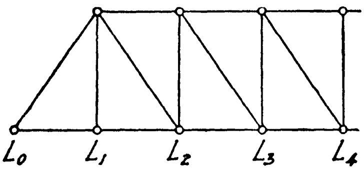
1	426	411	381	351	321	291	271.5	252	232.5	213	198	168	138	108	78	58.5	39	19.5
2	15	45	75	105	135	154.5	174	193.5	213	228	258	288	318	348	367.5	387	406.5	426
3	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
4	15	30	30	30	30	19.5	19.5	19.5	19.5	15	30	30	30	30	19.5	19.5	19.5	19.5
5	8'	5'	5'	5'	9'	5'	6'	5'	8'	8'	5'	5'	5'	9'	5'	6'	5'	5'
6	8'	13'	18'	23'	32'	37'	43'	48'	56'	64'	69'	74'	79'	88'	93'	99'	104'	109'
7	109'	101'	96'	91'	86'	77'	72'	66'	61'	53'	45'	40'	35'	30'	21'	16'	10'	5'
8	24550	22910	19880	17000	14270	11690	10190	8790	7500	6310	5514	4164	2965	1914	1014	605	292.5	97.5
9	22420	20860	17980	15250	12670	10240	8830	7530	6340	5240	4524	3325	2275	1374	624	312	97.5	
10	20380	18900	16170	13590	11160	8880	7570	6360	5270	4280	3632	2580	1682	932	331.5	117	97.5	
11	18060	16670	14120	11720	9470	7370	6180	5090	4110	3230	2678	1808	1088	518	97.5	117	331.5	
12	16220	14900	12500	10250	8155	6200	5110	4120	3240	2460	1980	1260	690	270	97.5	312	624	
13	13090	11910	9780	7800	5970	4290	3370	2550	1850	1245	900	450	150	175.5	448.5	838.5	1326	
14	11500	10400	8410	6580	4900	3370	2555	1830	1227	720	450	150	150	423	793.5	1281	1866	
15	10060	9030	7200	5520	3990	2605	1885	1262	755	345	150	150	450	820.5	1288.5	1873	2556	
16	8770	7810	6130	4600	3220	1992	1368	842	432	120	150	450	900	1368	1933.5	2620	3396	
17	6950	6110	4670	3380	2240	1248	780	409.5	156	240	630	1170	1860	2484	3205.5	4044	4980	
18	5240	4524	3325	2275	1374	624	312	97.5										
19	4280	3632	2580	1682	932	331.5	117											
20	3230	2678	1808	1088	518	97.5												
21	2460	1980	1260	690	270													
22	1245	900	450	150														
23	720	450	150															
24	345	150																
25	120																	

Uniform Load 3000# per lin. Ft.

MOMENT TABLE
COOPER'S E-60 LOADING
TWO 213 TON ENGINES + 6000 LBS. PER FOOT.
MOMENT IN THOUSAND FOOT POUNDS FOR ONE RAIL.
LOADS IN THOUSANDS OF POUNDS FOR ONE RAIL.

TABLE VI.

MAXIMUM SHEARS IN TRUSS BRIDGES FOR COOPER'S E50 LOADING.



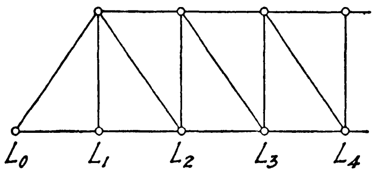
SHEARS FOR THROUGH SPANS
COOPER'S E·50 LOADING

Shears in Thousands of Pounds for One Rail

Number of Panels in Bridge	Panels	Length of Panel													
		12'0"	12'6"	13'0"	13'6"	14'0"	14'6"	15'0"	15'6"	16'0"	16'6"	17'0"	17'6"	18'0"	18'6"
3	L_0L_1	51·6	53·0	54·3	55·9	57·4	58·7	60·0	61·5	63·0	64·3	65·6	66·9	68·2	69·5
4	L_0L_1	71·6	73·6	75·5	77·6	79·6	81·6	83·6	85·5	87·3	89·0	90·6	92·6	94·5	96·4
	L_1L_2	34·4	35·6	36·7	37·7	38·6	39·6	40·6	41·7	42·7	43·9	45·0	46·1	47·2	48·3
	L_2L_3	7·9	8·4	8·9	9·4	9·8	10·3	10·7	11·2	11·7	12·2	12·7	13·1	13·5	13·9
5	L_0L_1	89·2	91·4	93·6	96·4	99·2	102·3	105·4	108·6	111·8	115·1	118·3	121·5	124·6	127·5
	L_1L_2	53·8	55·5	57·1	58·7	60·3	61·9	63·4	64·8	66·2	67·7	69·1	70·8	72·4	74·0
	L_2L_3	25·9	26·9	27·8	28·7	29·5	30·4	31·2	32·0	32·8	33·6	34·3	35·1	35·8	36·6
6	L_0L_1	106·7	110·5	114·3	118·7	123·1	127·1	131·0	134·9	138·8	142·7	146·5	150·2	153·8	157·5
	L_1L_2	72·1	74·2	76·3	78·1	79·8	82·2	84·6	86·9	90·1	93·0	95·8	98·5	101·1	103·6
	L_2L_3	43·4	44·9	46·3	47·7	49·1	50·4	51·7	52·9	54·0	55·3	56·5	57·6	58·6	59·7
	L_3L_4	20·2	21·1	21·9	22·6	23·3	24·1	24·8	25·6	26·3	27·0	27·6	28·3	28·9	29·6
7	L_0L_1	127·5	132·0	136·5	141·4	146·2	150·9	155·5	160·1	164·6	169·0	173·3	177·5	181·6	185·7
	L_1L_2	89·0	92·0	95·0	98·8	102·6	106·1	109·6	113·0	116·4	119·7	123·1	126·4	129·6	132·8
	L_2L_3	59·6	62·0	64·3	65·9	67·4	69·3	71·1	73·1	75·0	77·4	79·7	82·1	84·4	86·6
	L_3L_4	36·1	37·4	38·6	39·8	41·0	42·2	43·4	44·4	45·4	46·5	47·5	48·5	49·4	50·4
	L_4L_5	16·1	16·9	17·7	18·4	19·0	19·7	20·3	21·0	21·6	22·2	22·8	23·4	24·0	24·6
8	L_0L_1	147·2	152·3	157·4	162·9	168·4	173·6	178·8	183·8	188·7	193·6	198·4	203·1	207·8	212·5
	L_1L_2	108·4	112·6	116·7	121·0	125·3	129·5	133·7	137·8	141·8	145·7	149·5	153·2	156·9	160·5
	L_2L_3	76·8	79·5	82·2	85·0	87·8	90·9	93·9	96·8	99·6	102·6	105·6	108·5	111·4	114·2
	L_3L_4	52·0	53·7	55·3	56·7	58·1	59·8	61·4	63·1	64·8	66·7	68·5	70·4	72·2	74·0
	L_4L_5	30·5	31·7	32·8	33·9	35·0	36·1	37·1	38·0	38·9	39·9	40·9	41·7	42·5	43·4
	L_5L_6	13·1	13·8	14·5	15·1	15·7	16·4	17·0	17·6	18·1	18·7	19·2	19·8	20·3	20·8
9	L_0L_1	166·4	172·0	177·6	183·5	189·4	195·1	200·9	206·4	211·8	217·5	222·7	228·0	233·2	238·4
	L_1L_2	128·2	132·9	137·5	142·5	147·4	152·1	156·8	161·3	165·7	170·1	174·5	178·8	183·0	187·2
	L_2L_3	95·4	99·2	102·9	106·4	109·8	112·9	116·6	120·4	124·1	127·6	131·0	134·4	137·7	141·0
	L_3L_4	67·4	69·8	72·2	74·8	77·3	80·1	82·7	85·2	87·6	90·1	92·5	94·9	97·3	99·9
	L_4L_5	45·3	46·8	48·3	49·6	50·8	52·4	53·8	55·4	56·9	58·6	60·2	61·9	63·5	65·3
	L_5L_6	26·2	27·3	28·3	29·3	30·3	31·3	32·3	33·1	33·9	34·8	35·7	36·5	37·2	38·0

TABLE VII.

Maximum Shears in Truss Bridges for Cooper's E50 Loading.



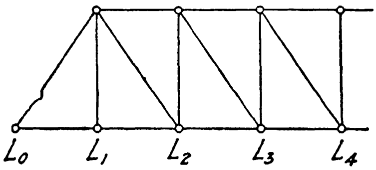
SHEARS FOR THROUGH SPANS
COOPER'S E·50 LOADING

Shears in Thousands of Pounds for One Rail.

Number of Panels in Bridge	Panels	Length of Panel													
		19'0"	19'6"	20'0"	20'6"	21'0"	21'6"	22'0"	22'6"	23'0"	23'6"	24'0"	24'6"	25'0"	25'6"
3	L_0L_1	70·8	72·0	73·2											
4	L_0L_1	98·2	100·7	103·0	105·6	108·2	110·7	113·2	115·5	117·7	120·0	122·2	124·4	126·5	128·7
	L_1L_2	49·3	50·3	51·3	52·2	53·1	54·0	54·9	55·8	56·7	57·4	58·2	59·0	59·7	60·5
	L_2L_3	14·3	14·7	15·0	15·3	15·6	15·9	16·2	16·5	16·7	17·0	17·2	17·5	17·8	18·1
5	L_0L_1	130·4	133·5	136·6	139·8	142·9	146·0	149·0	152·0	154·9	157·8	160·5	163·3	166·0	168·8
	L_1L_2	75·6	77·4	79·1	80·9	82·6	84·4	86·1	88·0	89·9	91·7	93·5	95·1	96·6	98·3
	L_2L_3	37·3	38·1	38·8	39·6	40·3	40·9	41·6	42·3	42·9	43·7	44·3	45·0	45·5	46·3
6	L_0L_1	161·1	164·6	168·1	171·7	175·2	178·8	182·3	185·8	189·2	192·6	195·9	199·2	202·5	205·9
	L_1L_2	106·1	108·6	111·0	113·6	116·0	118·5	120·8	123·2	125·4	127·9	130·1	132·4	134·5	136·8
	L_2L_3	60·7	62·1	63·5	65·1	66·6	68·2	69·6	71·3	72·9	74·5	75·9	77·4	78·6	80·2
	L_3L_4	30·2	30·8	31·4	32·1	32·8	33·4	34·0	34·5	35·0	35·5	36·0	36·6	37·1	37·6
7	L_0L_1	189·7	193·9	197·8	201·7	205·5	209·6	213·7	217·9	221·8	225·8	229·7	233·6	237·4	241·4
	L_1L_2	135·9	139·0	142·0	145·0	147·9	150·9	153·7	156·1	159·3	162·1	164·8	167·6	170·3	173·2
	L_2L_3	88·8	91·0	93·1	95·4	97·5	99·6	101·6	103·8	105·8	107·9	109·8	111·8	113·6	115·6
	L_3L_4	51·3	52·4	53·4	54·5	55·5	56·7	57·8	59·3	60·6	62·1	63·4	64·7	65·8	67·1
	L_4L_5	25·1	25·7	26·3	26·9	27·4	28·0	28·5	29·0	29·4	29·9	30·3	30·8	31·3	31·8
8	L_0L_1	217·1	221·7	226·3	230·8	235·2	239·9	244·3	248·9	253·4	258·0	262·5	267·1	271·5	276·0
	L_1L_2	164·1	167·7	171·3	174·8	178·2	181·7	185·0	188·4	191·7	195·1	198·3	201·7	204·9	208·3
	L_2L_3	117·0	119·8	122·5	125·1	127·6	130·5	132·9	135·4	137·8	140·3	142·7	145·2	147·5	150·0
	L_3L_4	75·8	77·8	79·8	81·7	83·6	85·5	87·3	89·2	91·0	92·8	94·5	96·3	98·0	99·8
	L_4L_5	44·2	45·2	46·1	47·1	48·0	49·0	49·9	51·0	52·1	53·1	54·1	55·3	56·4	57·4
	L_5L_6	21·3	21·9	22·4	22·9	23·4	23·9	24·4	24·9	25·3	25·7	26·0	26·5	26·9	27·3
9	L_0L_1	243·6	248·8	253·9	259·0	264·0	269·2	274·2	279·4	284·5	289·7	294·9	299·9	304·9	310·0
	L_1L_2	191·4	195·4	199·5	203·5	207·5	211·5	215·6	219·4	223·3	227·2	231·0	234·9	238·8	242·8
	L_2L_3	144·2	147·4	150·6	153·8	156·9	160·0	163·0	166·0	169·0	172·0	175·0	177·9	180·8	183·8
	L_3L_4	102·4	104·9	107·3	109·7	112·0	114·3	116·6	118·9	121·1	123·4	125·5	127·8	129·9	132·0
	L_4L_5	67·0	68·6	70·1	71·7	73·3	74·9	76·4	78·0	79·5	81·2	82·8	84·3	85·8	87·4
	L_5L_6	38·7	39·6	40·4	41·3	42·1	43·0	43·9	44·9	45·8	46·7	47·6	48·6	49·6	50·6

TABLE VIII.

MAXIMUM SHEARS IN TRUSS BRIDGES FOR COOPER'S E50 LOADING.



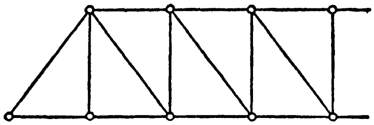
SHEARS FOR THROUGH SPANS
COOPER'S E·50 LOADING

Shears in Thousands of Pounds For One Rail.

Number of Panels in Bridge	Panels	Length of Panel													
		26'-0"	26'-6"	27'-0"	27'-6"	28'-0"	28'-6"	29'-0"	29'-6"	30'-0"	31'-0"	32'-0"	33'-0"	34'-0"	35'-0"
3	L_0L_1														
4	L_0L_1	130·9	133·1	135·2	137·3	139·3	141·5	143·6	145·8	147·9					
	L_1L_2	61·3	62·1	62·9	63·8	64·6	65·6	66·5	67·4	68·3					
	L_2L_3	18·4	18·6	18·9	19·1	19·3	19·6	19·8	20·1	20·3					
5	L_0L_1	171·4	174·1	176·7	179·4	181·9	184·5	187·0	189·6	192·0	197·1	202·4	207·5	212·6	217·6
	L_1L_2	100·1	101·9	103·6	105·4	107·1	108·9	110·6	112·3	114·0	117·3	120·3	123·5	126·5	129·5
	L_2L_3	46·9	47·7	48·3	49·0	49·6	50·5	51·3	52·1	52·8	54·3	55·8	57·3	59·1	60·8
6	L_0L_1	209·0	212·2	215·4	218·6	221·8	224·9	228·0	231·1	234·2	240·3	246·6	252·8	259·1	265·3
	L_1L_2	139·0	141·3	143·5	145·8	148·0	150·3	152·4	154·6	156·7	160·8	165·1	169·3	173·3	177·3
	L_2L_3	81·5	83·0	84·3	85·7	87·0	88·4	89·6	91·1	92·4	95·0	97·5	100·0	102·5	105·1
	L_3L_4	38·1	38·6	39·1	39·6	40·0	40·5	41·0	41·7	42·4	43·6	45·1	46·3	47·8	49·3
7	L_0L_1	245·2	249·1	252·8	256·6	260·4	264·1	267·7	271·4	275·0	282·3	289·6	297·1	304·6	312·0
	L_1L_2	175·9	178·8	181·5	184·4	187·0	189·9	192·5	195·4	197·9	203·3	208·5	213·8	218·8	224·0
	L_2L_3	117·4	119·3	121·1	123·0	124·8	126·6	128·3	130·2	131·9	135·3	138·8	142·5	146·0	149·6
	L_3L_4	68·3	69·6	70·8	72·0	73·1	74·3	75·4	76·7	77·8	80·1	82·4	84·5	86·6	88·8
	L_4L_5	32·1	32·6	33·0	33·5	33·8	34·3	34·6	35·1	35·6	36·5	37·5	38·5	39·8	41·0
8	L_0L_1	280·4	284·9	289·2	293·6	297·9	302·3	306·5	310·9	315·0	323·5	332·0	340·6	349·3	357·9
	L_1L_2	211·6	215·1	218·4	221·8	225·0	228·4	231·7	235·0	238·2	244·6	251·0	257·3	263·8	270·0
	L_2L_3	152·3	154·7	157·0	159·4	161·7	164·0	166·1	168·5	170·8	175·4	180·1	184·8	189·3	193·9
	L_3L_4	101·4	103·1	104·6	106·3	107·9	109·5	111·0	112·6	114·1	117·3	120·3	123·3	126·3	129·3
	L_4L_5	58·4	59·5	60·5	61·6	62·6	63·7	64·8	65·9	66·9	68·9	70·8	72·8	74·8	76·7
	L_5L_6	27·6	28·0	28·4	28·8	29·1	29·5	29·9	30·4	30·8	31·5	32·5	33·3	34·3	35·2
9	L_0L_1	315·0	320·1	325·0	330·0	334·9	339·9	344·7	349·7	354·5	364·1	373·8	383·5	393·5	403·3
	L_1L_2	246·7	250·6	254·5	258·5	262·4	266·3	270·2	274·0	277·8	285·4	293·0	300·5	308·0	315·5
	L_2L_3	186·7	189·6	192·4	195·3	198·0	200·9	203·8	206·7	209·5	215·3	221·0	226·8	232·5	238·2
	L_3L_4	134·1	136·3	138·4	140·5	142·5	144·6	146·6	148·6	150·6	154·8	158·8	162·7	166·6	170·5
	L_4L_5	88·9	90·4	91·8	93·3	94·8	96·2	97·6	99·0	100·4	103·1	105·8	108·6	111·3	114·0
	L_5L_6	51·5	52·4	53·3	54·2	55·0	55·9	56·8	57·6	58·4	60·3	62·0	63·8	65·5	67·2

TABLE IX.

Maximum Bending Moments in Pratt Truss Bridges for Cooper's E50 Loading.



L_0 L_1 L_2 L_3 L_4

BENDING MOMENTS FOR THROUGH SPANS
COOPER'S E-50 LOADING

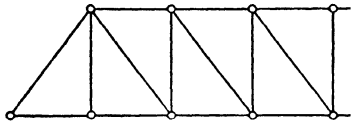
Moments in Thousands of Foot-Pounds for One Rail.

Number Panels in Bridge	Panel Point	Length of Panel											
		8'-0"	9'-0"	10'-0"	11'-0"	12'-0"	12'-6"	13'-0"	13'-6"	14'-0"	14'-6"	15'-0"	15'-6"
3	L_1	325	392	464	542	619	661	707	755	803	850	900	952
4	L_1	433	532	632	743	859	918	982	1 046	1 115	1 183	1 254	1 324
	L_2	569	681	821	964	1 110	1 189	1 269	1 352	1 441	1 529	1 624	1 720
5	L_1	540	662	792	929	1 071	1 140	1 217	1 298	1 389	1 480	1 580	1 679
	L_2	790	964	1 148	1 361	1 574	1 675	1 792	1 940	2 047	2 177	2 309	2 439
6	L_1	641	783	930	1 095	1 280	1 375	1 485	1 600	1 724	1 840	1 964	2 089
	L_2	1 008	1 166	1 465	1 710	1 997	2 135	2 289	2 445	2 616	2 792	2 984	3 174
	L_3	1 110	1 351	1 617	1 924	2 240	2 407	2 581	2 760	2 946	3 138	3 337	3 538
7	L_1	729	892	1 080	1 292	1 530	1 645	1 775	1 906	2 047	2 185	2 331	2 479
	L_2	1 215	1 475	1 748	2 070	.2 441	2 642	2 849	3 050	3 263	3 485	3 722	3 957
	L_3	1 425	1 739	2 086	2 465	2 879	3 100	3 332	3 560	3 802	4 040	4 312	4 595
8	L_1	815	1 021	1 254	1 500	1 766	1 900	2 047	2 200	2 358	2 516	2 680	2 845
	L_2	1 397	1 701	2 046	2 490	2 933	3 165	3 405	3 645	3 898	4 160	4 436	4 710
	L_3	1 715	2 100	2 529	2 991	3 498	3 775	4 078	4 383	4 710	5 040	5 380	5 720
	L_4	1 819	2 240	2 699	3 203	3 742	4 025	4 344	4 681	5 034	5 398	5 768	6 147
9	L_1	922	1 163	1 418	1 698	1 997	2 145	2 309	2 475	2 651	2 827	3 010	3 195
	L_2	1 576	1 955	2 404	2 888	3 400	3 670	3 946	4 224	4 511	4 804	5 107	5 420
	L_3	1 933	2 435	2 986	3 571	4 194	4 531	4 886	5 241	5 616	5 993	6 390	6 790
	L_4	2 127	2 598	3 186	3 860	4 588	4 970	5 370	5 770	6 186	6 610	7 047	7 485

		16'-0"	16'-6"	17'-0"	17'-6"	18'-0"	18'-6"	19'-0"	19'-6"	20'-0"	20'-6"	21'-0"	21'-6"
3	L_1	1 008	1 060	1115	1 170	1 228	1 285	1 346	1 404	1 464			
4	L_1	1 396	1 463	1 539	1 614	1 701	1 776	1 868	1 958	2 061	2 166	2 273	2 380
	L_2	1 819	1 923	2 029	2 134	2 240	2 349	2 465	2 581	2 701	2 821	2 946	3 074
5	L_1	1 788	1 895	2 009	2 123	2 242	2 355	2 477	2 600	2 731	2 864	3 001	3 138
	L_2	2 580	2 724	2 880	3 030	3 190	3 350	3 518	3 685	3 943	4 144	4 347	4 555
6	L_1	2 220	2 351	2 488	2 626	2 769	2 910	3 062	3 210	3 362	3 516	3 678	3 840
	L_2	3 372	3 569	3 775	3 978	4 194	4 415	4 650	4 885	5 255	5 501	5 750	5 998
	L_3	3 742	3 952	4 170	4 422	4 681	4 948	5 215	5 487	5 746	6 028	6 321	6 617
7	L_1	2 633	2 786	2 945	3 104	3 268	3 434	3 605	3 778	3 955	4 130	4 317	4 505
	L_2	4 203	4 450	4 705	4 958	5 218	5 480	5 746	6 025	6 326	6 613	6 914	7 215
	L_3	4 898	5 200	5 509	5 815	6 135	6 460	6 800	7 140	7 646	7 990	8 347	8 710
8	L_1	3 018	3 189	3 372	3 553	3 741	3 930	4 125	4 320	4 525	4 727	4 939	5 150
	L_2	4 994	5 280	5 576	5 873	6 180	6 487	6 806	7 125	7 458	7 805	8 162	8 520
	L_3	6 072	6 430	6 806	7 180	7 573	7 985	8 369	8 780	9 234	9 630	10 070	10 515
	L_4	6 516	6 915	7 331	7 740	8 164	8 595	9 043	9 490	9 943	10 396	10 862	11 317
9	L_1	3 388	3 582	3 785	3 987	4 198	4 410	4 629	4 850	5 079	5 308	5 545	5 780
	L_2	5 747	6 074	6 414	6 755	7 108	7 463	7 830	8 198	8 578	8 970	9 378	9 790
	L_3	7 204	7 620	8 054	8 496	8 959	9 415	9 892	10 372	10 880	11 375	11 900	12 425
	L_4	7 966	8 460	8 980	9 490	10 010	10 530	11 065	11 605	12 172	12 735	13 310	13 880

TABLE X.

MAXIMUM BENDING MOMENTS IN PRATT TRUSS BRIDGES FOR COOPER'S E50 LOADING.



L_0 L_1 L_2 L_3 L_4

BENDING MOMENTS FOR THROUGH SPANS COOPER'S E·50 LOADING

Moments in Thousands of Foot-Pounds for One Rail.

Number Panels in Bridge	Panel Point	Length of Panel											
		22'-0"	22'-6"	23'-0"	23'-6"	24'-0"	24'-6"	25'-0"	25'-6"	26'-0"	26'-6"	27'-0"	27'-6"
3	L_1												
4	L_1	2490	2597	2708	2819	2933	3046	3163	3282	3402	3526	3649	3774
	L_2	3205	3338	3470	3607	3743	3883	4025	4170	4344	4501	4681	4858
5	L_1	3278	3418	3562	3705	3852	3999	4150	4301	4456	4611	4770	4929
	L_2	4767	4978	5193	5415	5640	5865	6093	6371	6552	6783	7014	7250
6	L_1	4008	4175	4349	4522	4700	4878	5061	5245	5433	5622	5816	6010
	L_2	6250	6501	6756	7011	7270	7525	7794	8068	8352	8654	8960	9268
	L_3	6921	7228	7538	7850	8166	8491	8821	9153	9490	9828	10170	10514
7	L_1	4702	4897	5100	5303	5512	5721	5936	6051	6373	6595	6823	7051
	L_2	7530	7845	8173	8503	8842	9182	9530	9875	10236	10600	10980	11357
	L_3	9079	9448	9826	10207	10609	11017	11444	11870	12312	12752	13203	13653
8	L_1	5373	5594	5829	6061	6300	6540	6787	7035	7289	7540	7806	8069
	L_2	8890	9260	9640	10030	10430	10832	11244	11655	12080	12508	12950	13392
	L_3	10993	11475	11976	12472	12981	13490	14010	14528	15063	15605	16163	16718
	L_4	11805	12283	12790	13289	13795	14300	14820	15340	15875	16413	16965	17514
9	L_1	6030	6280	6542	6804	7074	7344	7622	7900	8188	8477	8774	9070
	L_2	10216	10640	11082	11525	11985	12448	12925	13400	13890	14380	14888	15400
	L_3	12978	13535	14118	14705	15308	15910	16528	17145	17778	18414	19070	19730
	L_4	14472	15068	15684	16300	16930	17560	18205	18850	19515	20180	20870	21557

		28'0"	28'6"	29'0"	29'6"	30'0"	31'0"	32'0"	33'0"	34'0"	35'0"	36'0"	37'0"
3	L_1												
4	L_1	3900	4031	4165	4300	4436							
	L_2	5034	5215	5398	5580	5768							
5	L_1	5092	5255	5422	5589	5760	6113	6477	6849	7229	7617		
	L_2	7492	7736	7984	8232	8482	8985	9496	10012	10591	11192		
6	L_1	6208	6408	6612	6817	7026	7449	7891	8346	8812	9288		
	L_2	9580	9897	10218	10547	10880	11557	12248	12978	13728	14510		
	L_3	10862	11208	11565	11925	12296	13040	13796	14563	15341	16145		
7	L_1	7286	7521	7762	8003	8250	8751	9267	9805	10356	10920		
	L_2	11742	12125	12520	12918	13330	14164	15016	15894	16810	17755		
	L_3	14112	14571	15039	15507	15984	16965	17963	18979	20012	21073		
8	L_1	8338	8608	8887	9165	9450	10029	10622	11239	11874	12525	13190	13873
	L_2	13850	14308	14780	15250	15730	16721	17732	18768	19850	20959	22092	23247
	L_3	17285	17852	18431	19010	19600	20812	22052	23312	24601	25921	27271	28652
	L_4	18075	18635	19210	19795	20406	21635	22895	24197	25530	26905	28311	29726
9	L_1	9376	9686	9996	10310	10633	11289	11962	12656	13376	14114	14871	15644
	L_2	15930	16460	17005	17547	18100	19244	20416	21616	22855	24144	25425	26793
	L_3	20405	21080	21770	22461	23168	24605	26081	27595	29133	30710	32327	33983
	L_4	22260	22955	23678	24405	25170	26707	28282	29908	31572	33289	35051	36826

SHEARS AND MOMENTS IN A PLATE GIRDER BRIDGE.—The maximum shears and moments in an 86 ft. span deck girder railway bridge are shown in Fig. 20. In calculating the maximum live load shears the girder was divided into sections about 7 ft. in length and the maximum shears were calculated as in a truss bridge. The maximum bending moments were also calculated for the same points in the girder. The make-up of the tension flange and the rivet spacing is shown in Fig. 20.

The stress diagram for a 60 ft. span single track deck plate girder bridge is shown in Fig. 21.

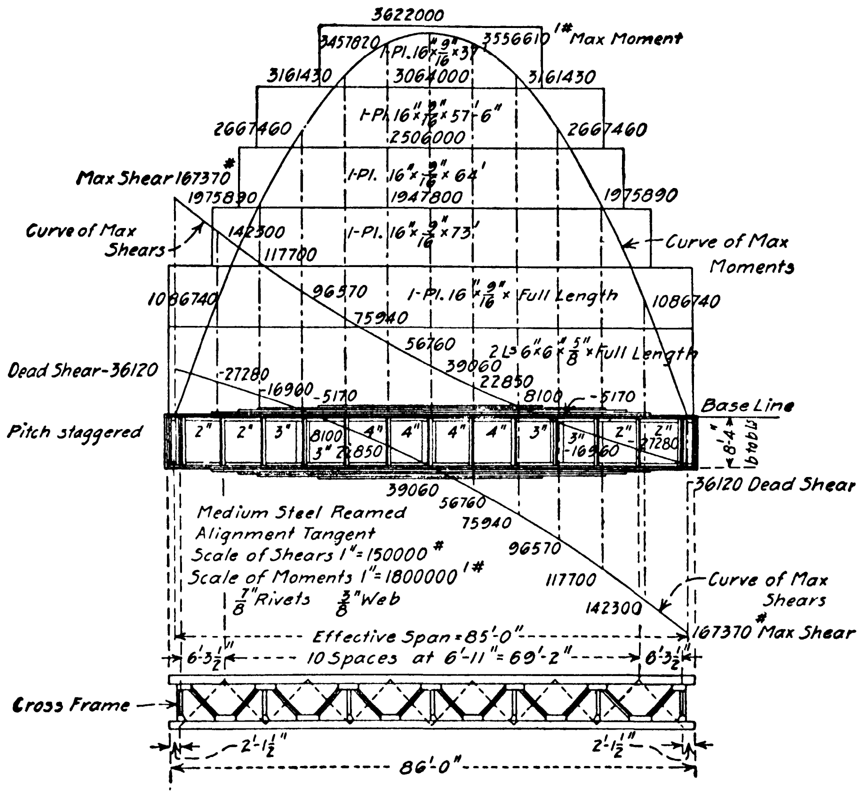


FIG. 20. SHEARS AND MOMENTS IN A RAILWAY PLATE GIRDER.

MATERIAL.—Open-hearth carbon steel complying with the specifications of the Am. Ry. Eng. Assoc. as given in the last part of this chapter is commonly used for bridges up to spans of 500 to 550 feet. For spans of more than 500 or 550 feet to about 650 feet carbon and nickel steel are used, or nickel steel alone is used. For spans of 650 to 750 feet nickel steel alone should be used. For an exhaustive discussion of the use of nickel steel in the construction of bridges see article entitled "Nickel Steel for Bridges" by Mr. J. A. L. Waddell, M. Am. Soc. C. E., in Trans. Am. Soc. C. E., Vol. 63, 1909. An excellent discussion of the design of large bridges is given in "Design of Large Bridges with Special Reference to the Quebec Bridge" by Ralph Modjeski, Consulting Engineer, in Journal Franklin Institute, September, 1913.

ALLOWABLE STRESSES.—The allowable stresses on carbon steel as adopted by the Am. Ry. Eng. Assoc. are given in the specifications in the last part of this chapter. Out of 39 railroads in the United States 24 were using the Am. Ry. Eng. Assoc. specifications for allowable unit stresses in 1913. For additional data on unit stresses, see Table XVI.

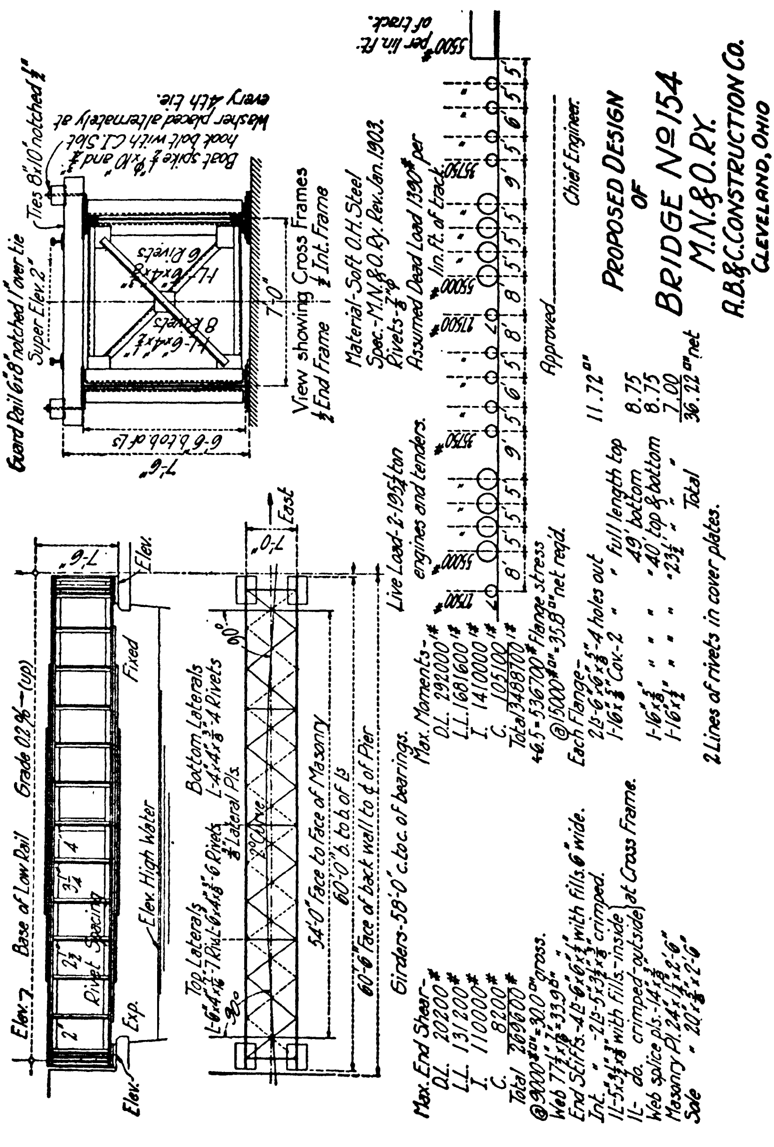


FIG. 21. STRESS SHEET FOR DECK PLATE GIRDER SPAN.

ECONOMIC DESIGN OF RAILWAY BRIDGES.—Pin-connected truss bridges have been used for railroads on account of the ease of erection, ease in calculating the stresses, and the simplicity of details which give small secondary stresses. The present practice in railway bridge design is to use plate girders for spans up to about 115 ft., and riveted truss bridges for longer spans; pin-connected bridges being used only for very long spans and for spans of 200 ft. and over where there is some special reason such as ease of erection or low cost. The author would recommend pin-connected truss bridges for all spans of 200 ft. and over for the following reasons:—(1) the weight of a pin-connected truss bridge with eye-bars is less than the weight of a riveted truss bridge of the same span and capacity, and while the shop cost per pound of pin-connected truss

bridges is slightly higher than for riveted truss bridges, the total cost erected of the structural steel in the pin-connected bridge is less than the steel in the riveted bridge. (2) The pin-connected truss bridge can be erected in less time at a very much less cost than the riveted truss bridge. (3) The secondary stresses in the pin-connected truss bridge are smaller than in the riveted truss bridge and the structure is more efficient. (4) With the present ballasted floors the vibration and impact stresses are no greater in a pin-connected truss bridge than in a riveted truss bridge. Riveted tension members are difficult to design and are expensive of material and labor. Eyebars are ideal tension members in which the material is used efficiently. For the above reasons the author predicts that the pin-connected bridge for spans of 200 ft. and over will regain its place as a standard type of railroad bridge.

The Pratt truss with parallel chords is used for pin-connected spans up to about 250 ft., while riveted truss spans are made with Pratt or Warren trusses; double and triple intersection trusses are also used for riveted trusses. For long span bridges the subdivided Pratt truss with inclined chords (Petit truss) is generally used. The width center to center of trusses should not be less than one-twentieth of the span, and preferably not less than one-eighteenth. The height at the center should be from one-fifth to one-seventh of the span; the Municipal Bridge at St. Louis has a center height of one-sixth of the span. The height at the ends should be only sufficient for an effective portal. The most economical inclination of diagonals is very nearly 40 degrees, so that in a Petit truss the panel length should be about 0.42 times the height. For the most economical web system the panels should vary in length as the depth varies, but this increases the weight of the floor and also increases the shop cost and cost of erection, so that constant panel lengths are commonly used. One railroad specification requires that panel lengths shall not exceed 35 feet. For truss bridges of the Pratt type with two stringers and an open timber floor the present practice is to use a panel length of $22\frac{1}{2}$ to $27\frac{1}{2}$ ft., with 25 ft. as an average. Increasing the length of the panels increases the weight of the floor system, and decreases the weight of the trusses. The economical panel lengths for bridges with ballasted floor is less than for bridges with open timber floor. Riveted truss bridges with triple-intersection web members, Fig. 41, are made with very short panels.

With the increase in the size of the sections in a bridge great care must be taken in detailing to use details that will develop the full strength of the members. Increased details increase the shop cost and for this reason there is a tendency for bridge companies to cut down details and to change details so as to simplify shop work even at the expense of added weight in order to obtain a low pound price. For this reason detail drawings, not necessarily shop drawings, should always be made by the designing engineer. The author has in mind a case where to change the details of a plate girder so that multiple punches might be used required the addition of details equal to 5 per cent of the weight of the span and the addition of 25 per cent to the number of field rivets, with no increase in efficiency. It is needless to say the change was not made.

An empirical rule for calculating the economical depth of plate girder spans is to *make the area of the flanges equal to the area of the webs.* The actual depths of plate girders are commonly slightly less than the depth given by the above rule. The minimum thickness of $\frac{3}{8}$ inch for plate girder webs should be used only for stringers with short spans, and the thickness of the web should be increased as the span and depth of the girder increases. For the depths and spacing of plate girders designed under Common Standard Specifications 1006, see Table I.

DETAILS OF RAILWAY BRIDGES.—It is very important that the details of railway bridges be worked out with great care. A few standard details will be briefly described.

Sections for Chords and Posts.—Chord sections are shown in (a) to (i) in Fig. 22. Sections (a) and (b) are used for light chords and (c), (d) and (e) for heavy chords. Sections (a) and (d) are also made by turning the angles in, as in section (i). Sections (f) to (i) are used for chord sections, for intermediate posts and for columns. Sections (n) and (p) to (t) are used for column sections. Chord sections, posts and columns with diaphragms or webs at right angles to each other as in (a) to (e), (n), and (p) to (t) give much better results under actual service than laced sections as in (f) to (i) and (o). Sections (j) to (m) and (o) are used for struts and braces.

Floors.—Bridges may have open timber floors as in Fig. 23, or ballasted floors as in Fig. 24, or in Fig. 25. For track elevation and for bridges crossing over streets, buildings, and similar locations and for ballasted floors, the bridge floor is waterproofed and the water falling on the floor is carried to the ground through properly arranged drains.

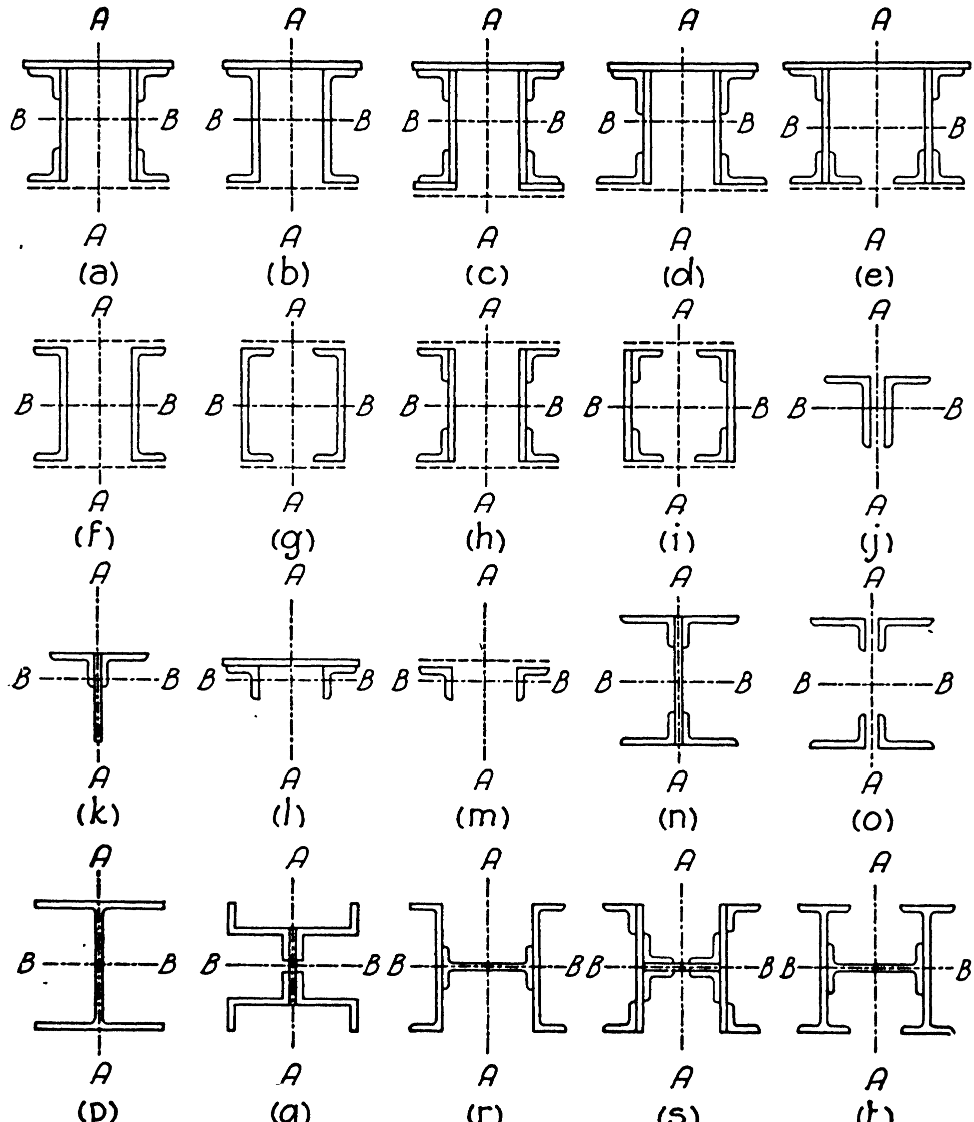


FIG. 22. TYPES OF COLUMNS AND TOP CHORD SECTIONS.

Details of the standard timber floors used by the Southern Pacific R. R., the Union Pacific R. R. and other Harriman Lines are given in Fig. 23. For additional details of open timber floors see Fig. 1 and Fig. 2, Chapter VII. The American Railway Engineering Association in 1912

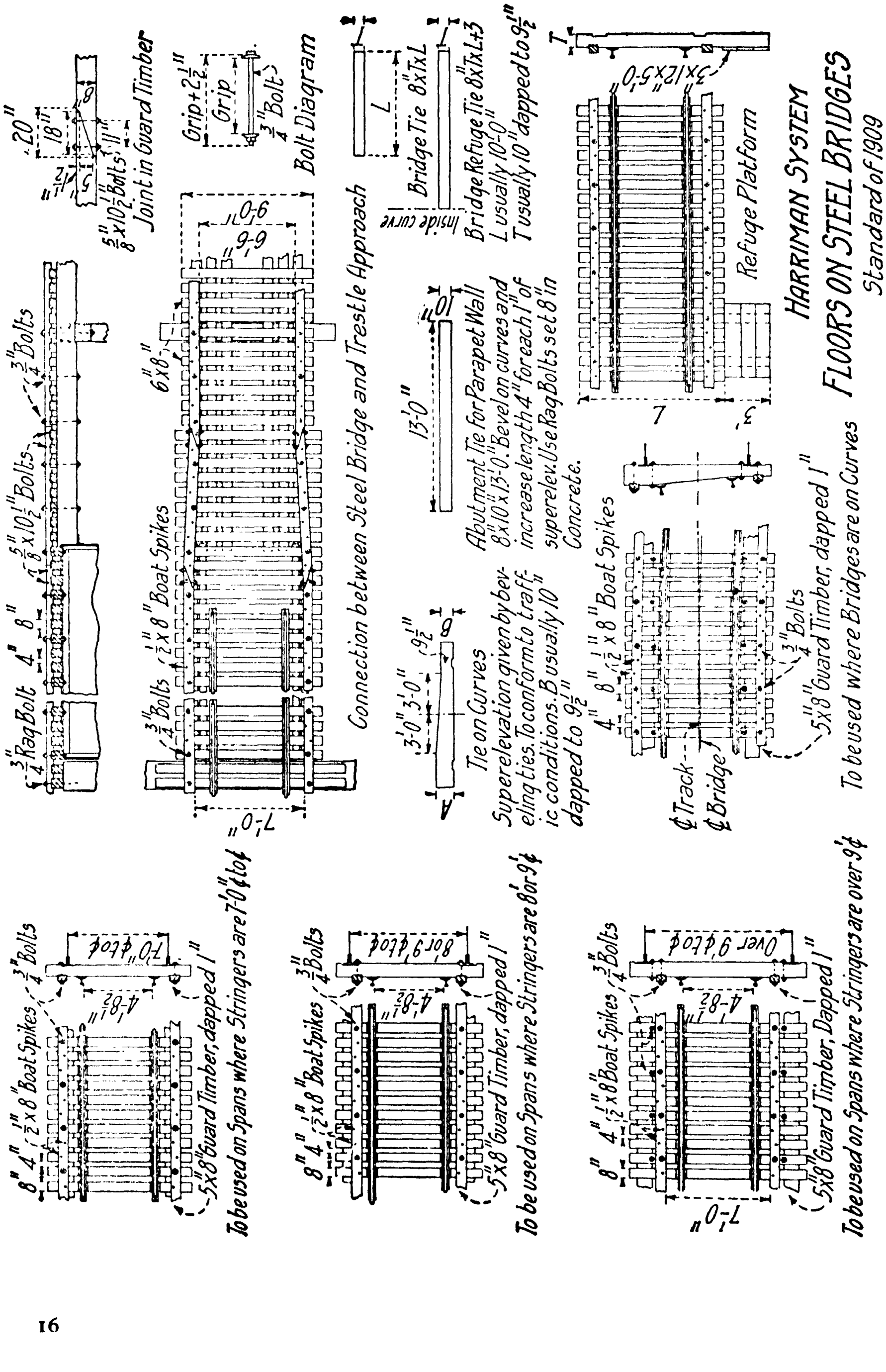


FIG. 23. TIMBER FLOORS FOR RAILWAY BRIDGES.

recommended that guard timbers be used on all open-floor bridges, also that guard rails be used on all bridges, and that the guard rails should extend at least 50 ft. beyond the end of the bridge. For additional details see Chapter VII, "Timber Bridges and Trestles."

Details of a ballasted floor with a reinforced concrete slab deck, and a ballasted floor with a timber deck, as designed and used by the Chicago, Milwaukee & St. Paul Ry. are given in Fig. 24. The reinforced concrete slabs are made either at the bridge site or at some other convenient location and are hoisted into place after the concrete has gained sufficient strength.

The Chicago, Burlington & Quincy R. R. uses reinforced concrete slabs for a ballasted deck on deck girders that differ from the Chicago, Milwaukee & St Paul slabs in Fig. 24, in the following details. The reinforced concrete slabs are 14 ft long in place of 13 ft.; and are 5 ft. wide in place of 3 ft. 7 in. The top of the slabs and the edges of the slabs are painted with tar paint (made of 16 parts coal tar, 4 parts Portland cement, and 3 parts kerosene). The edges of the reinforced concrete slabs are beveled and after the slabs are laid the joint between the slabs is packed with oakum for a depth of 1 in. at the bottom and the remainder of the joint is filled with 1 to 3 Portland cement mortar. Where the reinforced concrete deck is placed on a deck girder with cover plates, a strip of No. 22 gage lead 3 in. wider than the cover plate is placed on top of the cover plate and forced down over the rivet heads. After the slabs have been put in place and blocked up to the proper elevation the space between the lead sheet and the slab is filled with 1 to 3 Portland cement mortar. The minimum thickness of the mortar joint is one inch. Cinders or slag are not used for ballast on reinforced concrete slab decks.

A standard reinforced concrete floor for a through plate girder bridge as designed by the Chicago, Burlington & Quincy R. R. is shown in Fig. 25. The concrete is 1 : 2 : 4 Portland cement concrete. The upper surface of the concrete slab is painted with coal tar paint, the same as the deck slabs. Zinc sheets, No. 22 gage and 8 in. wide are placed on the tops of the floorbeams.

A steel plate ballasted floor on a through riveted truss bridge is shown in Fig. 41.

WATERPROOFING BRIDGE FLOORS.—The problem of waterproofing bridge floors is a difficult one and has been worked out in great detail by the engineers of many railroads, and by the American Railway Engineering Association. For a very full discussion of the problem, see the proceedings of the American Railway Engineering Association, especially Volume 14, 1913, and Volume 15, 1914. The following extracts from the report of a committee of the American Railway Engineering Association presented at the annual meeting of the society in March, 1914, are of value.

The methods of waterproofing are stated as follows:—

"The ordinary methods of waterproofing are.

"(1) *Coatings:* (a) Linseed oil paints and varnishes. (b) Bituminous; asphalt and coal tar. (c) Liquid hydrocarbons. (d) Miscellaneous compounds. (e) Cement mortar.

"(2) *Membranes:* Felts and burlaps in combination with various cementing compounds.

"(3) *Integrals:* (a) Inert fillers. (b) Active fillers.

"(4) *Watertight concrete construction.*"

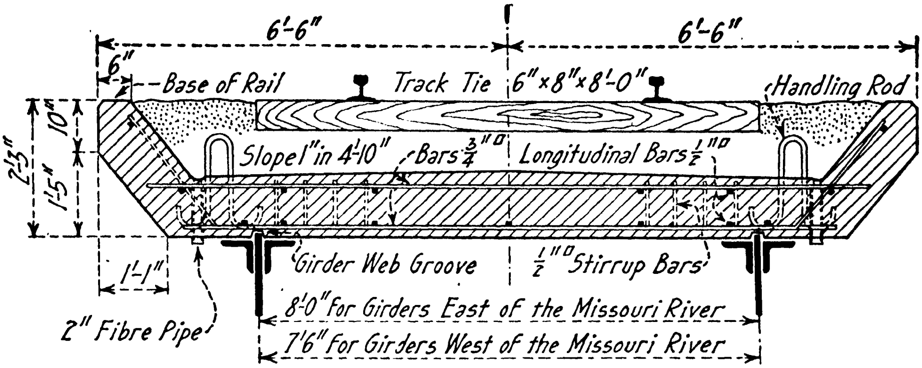
The conclusions reached in the report are as follows:—

"(1) Watertight concrete may be obtained by proper design, reinforcing the concrete against cracks due to expansion and contraction, using the proper proportions of cement and graded aggregates to secure the filling of the voids and employing proper workmanship and close supervision.

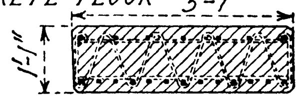
"(2) Membrane waterproofing, of either asphalt or pure coal tar pitch in connection with felts and burlaps, with proper number of layers, good materials and workmanship and good working conditions, *is recommended as good practice for waterproofing masonry, concrete and bridge floors.*

"(3) Permanent drainage of bridge floors is essential to secure good results in waterproofing.

"(4) Integral methods of waterproofing concrete have given good results. Special care is required to properly proportion the concrete, mix thoroughly and deposit properly so as to have the void-filling compounds do the required duty; if this is neglected the value of the compound is lost and its waterproofing effect is destroyed. Careful tests should be made to ascertain the proper proportions and effectiveness of such compounds. Integral compounds should be used with caution, ascertaining their chemical action on the concrete as well as their effect on its strength; *as a general rule, integral compounds are not to be recommended,* since the same results as to watertightness can be obtained by adding a small percentage of cement and properly grading the aggregate.



SECTION OF STANDARD CONCRETE FLOOR



SECTION AT CENTER OF TRACK

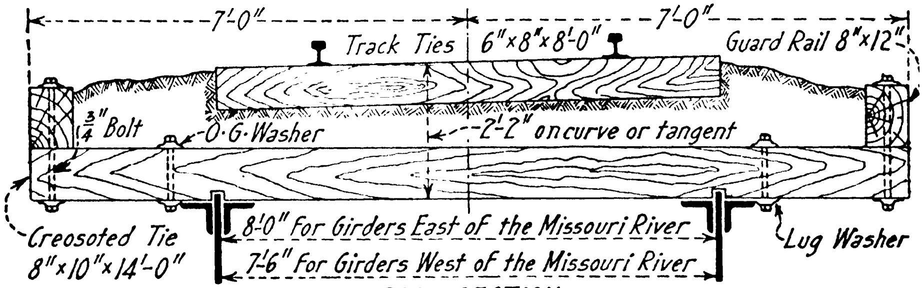
BILL OF MATERIAL FOR 3'-7" SLAB

	No.	Size	Length	Remarks
Bars	13	3/4"□	12'-0"	Bars "A" bent in bottom of slab
	7	3/4"□	11'-6"	Straight in top of slab.
	15	1/2"□	3'-3"	Longitudinal.
	8	1/2"□	10'-0"	Stirrup
	2	3/4"□	4'-9"	Handling Rods U.
	2	2"	1'-0"	Fibre Pipes, for all but end slab
	1.92			Cu. Yds. of Concrete

Weight of one Slab = 3.88 tons.

Weight of Floor Section—Concrete.

Ties 6"x8"x8'0", 15" ctrs. = 115# per lin. ft. of track
14.4 cu. ft. Concrete @ 145# = 2090# " " " "
10.8 cu. ft. Gravel @ 110# = 1190# " " " "
Reinforcing Steel = 110# " " " "
2-100# Rails = 65# " " " "
Total = 3570# per lin. ft. of track



CROSS SECTION

Weight of Floor Section—Timber

Track Ties, 15" centers = 115# per lineal foot of track
13.5 cu. ft. of Gravel @ 110# = 1485# " " " " "
Floor Ties @ 4½# B.M. = 630# " " " " "
Guard Rails @ 4½# B.M. = 70# " " " " "
2-100# Rails = 65# " " " " "
Total = 2365# per lineal foot of track

STANDARD TIMBER BALLAST FLOOR

FIG. 24. STANDARD BALLASTED FLOORS. CHICAGO, MILWAUKEE & ST. PAUL RY.

"(5) Surface coatings, such as cement mortar, asphalt or bituminous mastic, if properly applied to masonry reinforced against cracks produced by settlement, expansion and contraction, may be successfully used for waterproofing arches, abutments, retaining walls, reservoirs and similar structures; *for important work under high pressure of water these cannot be recommended for all conditions.*

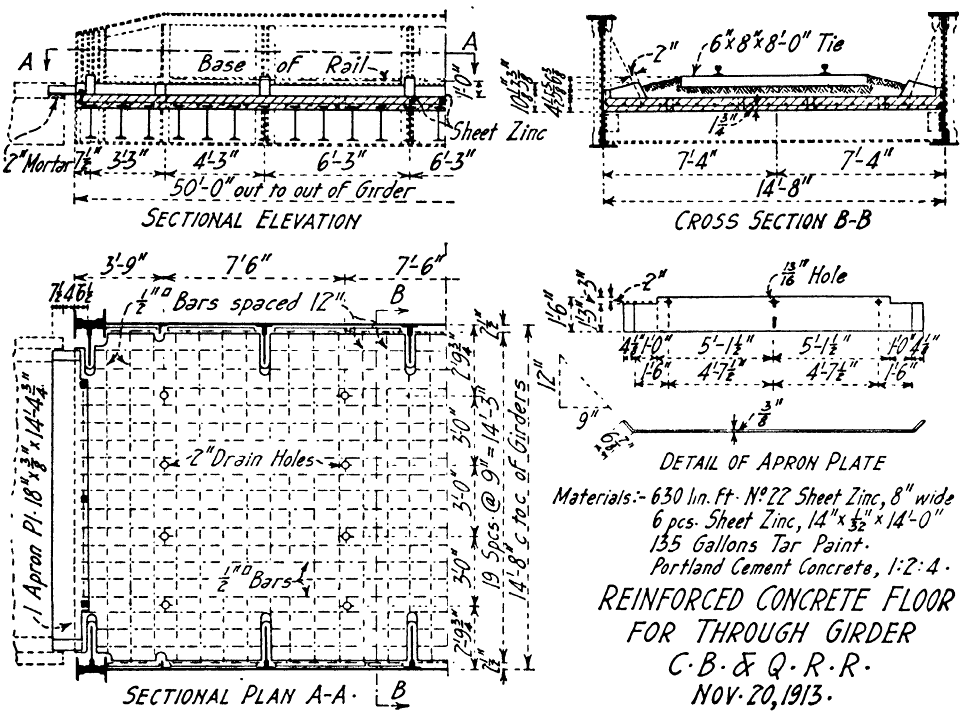


FIG. 25. REINFORCED CONCRETE FLOOR FOR THROUGH PLATE GIRDER BRIDGE. C. B. & Q. R. R.

"(6) Surface brush coatings, such as oil paints and varnishes, *are not considered reliable or lasting for waterproofing of masonry.*"

The membrane method of waterproofing bridge floors will be shown by describing the standard methods of waterproofing in use by two railroads.

CHICAGO, MILWAUKEE & ST. PAUL RY. SPECIFICATIONS FOR WATERPROOFING. The specifications of the Chicago, Milwaukee & St. Paul Ry. for waterproofing are as follows:

The necessary provision for drainage and expansion must be made in designing the structure. The waterproofing should never be compelled to resist hydrostatic pressure, and the membrane should always be protected by a layer of concrete.

(1) **Preliminary.**—Fill all openings and pockets in the concrete except expansion joints with cement mortar, and round off all sharp corners. Wherever waterproofing stops on a vertical surface the end should be flashed into a groove in the concrete.

(2) **Preparing the Surface.**—Thoroughly clean and dry the concrete surface using wire brushes and being careful to remove all the laitance. If necessary use hot sand to dry the concrete. Apply a coat of gasolene to the clean dry surface and follow with a coat of cold primer, spreading the primer evenly with a brush. Omit the primer where tar paper is to be placed and over expansion joints.

(3) **Laying the Burlap.**—After the primer coat has completely dried, apply a coat of pure hot asphalt, and mop until the layer has a thickness of ⅛ in. While the asphalt is still hot begin laying the burlap. Lay the first strip of burlap transverse to the drainage at the lowest point. Lay the strips shingle fashion, as for tar and gravel roofs, and parallel to the first strip working

up to the summit and exposing one-third of each width of burlap to the weather. Press each strip firmly into the asphalt, then mop well with pure melted asphalt taking care to thoroughly saturate the burlap and to fill all cracks and blow holes. Lap the joints in the strips 6 in. On this three-ply layer of burlap spread a continuous layer of hot asphalt mopping well until a layer of ⅜ in. is obtained. See (f) Fig. 26.

(4) **Summit Joints.**—After the work has been brought up to the desired point from both sides, interlap in order the strips which reach across the joint, mopping asphalt between burlap surfaces. Place a strip of burlap along the joint for a closing strip; and complete by laying the upper ⅜ in. of asphalt as before described. See (g) Fig. 26.

(5) **Longitudinal Joints.**—If possible the waterproofing should be laid in one run the full width transverse to the drain slope of the surface to be waterproofed. The ends of the burlap strips should be flashed into recesses in the walls, curbs or parapets as shown in (e) Fig. 26. Where longitudinal joints are necessary cut the burlap long enough to extend 12 in. beyond the primed and asphalted surface of the concrete and use care as the strips are laid that the 12 in. strip is kept free from asphalt. When the succeeding section is to be waterproofed fold back the projecting strips of burlap over the completed waterproofing and bring the new up against the completed portion of the waterproofing, interlapping the projecting ends of the burlap with the new burlap as the work progresses, (f) Fig. 26. On concrete trestle or subway slabs longitudinal joints in the waterproofing should preferably be on the center line of the slabs. If it is necessary to place joints in the waterproofing over joints in the slabs special care should be taken.

(6) **Expansion Joints.**—Lay two continuous strips of tar paper 36 in. wide over the expansion joint, being careful to see that no asphalt gets between or under the two strips of tar paper. Then mop the top strip with hot asphalt and carry the waterproofing over the top of the paper the same as if no joint existed. See (b) and (h) Fig. 26.

(7) **Concrete Protection.**—After the ⅜ in. layer of asphalt on top of the burlap has become cold, spread a ⅝ in. layer of concrete evenly over the surface. Then press a layer of expanded metal into the concrete, and cover the metal with a layer of concrete ½ in. thick making the total thickness of the concrete 1⅛ in., and trowel the concrete smooth. Protect the concrete from the sun for 24 hours after laying. The joints in the expanded metal should be lapped 6 in. See (d) Fig. 26.

(8) **Materials.**—*Burlap.*—The burlap is to be treated 8 oz. open mesh furnished in widths of 36 in. to 42 in.

Concrete.—The concrete is to be 1 part Portland cement, 2 parts torpedo sand, and 3 parts stone or gravel that will pass a ½ in. ring.

Mortar.—The mortar is to be 1 part Portland cement and 2 parts washed torpedo sand.

Primer.—The primer is made by pouring hot asphalt in 80 per cent gasolene until mixture will spread readily with a brush.

Asphalt.—Pure asphalt conforming to accepted specifications is to be used. Before using the asphalt heat it in a suitable kettle to a temperature not exceeding 450° F. The temperature is to be taken with a thermometer. Asphalt heated above 450 degrees F. or giving off yellow fumes is to be discarded as overheated.

Expanded Metal.—The expanded metal is to be equivalent to Northwestern Expanded Metal Co's. "2¼ in. No. 16 Regular" expanded metal.

Tar Paper.—The tar paper will be furnished in rolls 36 in. wide.

CHICAGO, BURLINGTON & QUINCY R. R. SPECIFICATIONS FOR WATERPROOFING.—The specifications of the Chicago, Burlington & Quincy R. R. for waterproofing are as follows:

(1) **Description.**—The waterproofing shall consist of a mat of 4-ply of burlap and 1-ply of felt thoroughly saturated and bonded together with waterproofing asphalt and covered with one inch of sand and asphalt mastic.

(2) **Preparing the Surface.**—The surface of the concrete shall be smooth, clean and dry. Upon this surface apply a coat of primer, which shall be thin enough to penetrate the concrete and form an anchorage for the waterproofing. *No waterproofing shall be done when the temperature is less than 60 degrees F.*

(3) **Applying the Burlap.**—After the priming coat has dried, a heavy coat of waterproofing asphalt heated to a temperature of 400 degrees F. shall be applied with mops the width of the burlap, and while the asphalt is still hot a layer of burlap shall be bedded in it. The burlap shall be laid just behind the mopping and shall be swept free from folds and pockets with a broom. The surface of the burlap shall be heavily mopped with waterproofing asphalt. Three more ply of burlap shall be laid in the same manner, making a 4-ply burlap mat all thoroughly saturated and bonded together.

The top of the burlap mat shall be heavily mopped with asphalt and one layer of felt saturated with asphalt shall be laid on the burlap and the edges of the felt lapped at least 3 inches and sealed with asphalt. The top of this felt shall also be mopped with waterproofing asphalt.

(4) **Mastic Protection.**—The burlap and felt mat shall be covered with one inch of asphalt mastic laid in one layer, the mastic to be composed of one part waterproofing asphalt and four

parts fine gravel graded from ¼ in. to fine sand. The top of the mastic shall be leveled off with wooden floats and mopped with waterproofing asphalt.

(5) **Expansion Joints.**—At all expansion joints in the concrete a fold to allow for the expansion of the structure shall be formed by laying the burlap and felt over a one-inch pipe; the pipe being removed as the mat is being completed.

(6) **Splices and Flashing.**—Where the work is stopped before being completed at least 3 feet of burlap at the end and one-half the width of the burlap at the side shall be left exposed to form a splice.

Special care shall be taken to seal the waterproofing at the sides and ends of the bridge. The burlap and mastic shall be carried up the parapet walls at the sides and the ends of the burlap shall be concreted into a recess in the walls so that no water can enter. The burlap shall be carried down over the back-walls at the ends of the bridge to cover all construction joints and shall run into a line of tile to facilitate the escape of the water.

(7) **Materials.**—*Burlap.*—The burlap is to be 8 oz. open mesh high grade burlap saturated with an asphalt meeting the specifications for waterproofing asphalt. It shall come in rolls which shall be placed on end for shipment and storage, and shall not stick together in the roll.

Felt.—The felt shall be a good quality of wool felt saturated and coated with an asphalt meeting the specifications for waterproofing asphalt. It shall come in rolls which shall be placed on end for shipment and storage, and shall not stick together in the roll. It shall not weigh less than 15 lb. per 100 sq. ft.

Primer.—The primer shall be an asphaltic compound of approved quality and capable of adhering firmly to the concrete.

Waterproofing Asphalt.—The waterproofing asphalt shall meet the following requirements.

1. The specific gravity of the asphalt desired shall be greater than 0.95 at 77 degrees F.
2. The flowing point shall not be less than 100 degrees F. nor more than 140 degrees F.
3. The flash point shall not be lower than 450 degrees F.

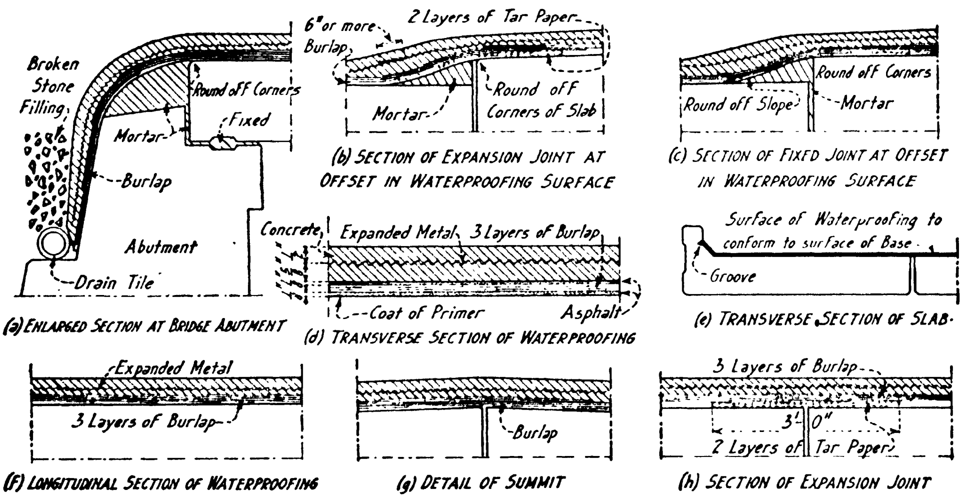


FIG. 26. STANDARD METHOD OF WATERPROOFING BRIDGE FLOORS. C. M. & ST. P. RY.

4. The penetration at 80 degrees F. for a period of 30 seconds shall be at least 15 millimeters and must not exceed 20 millimeters. This penetration to be measured with a Vicat needle weighing 300 grams, one end being one millimeter in diameter for a distance of 6 centimeters.
5. When heated to a temperature of 325 degrees F. for 7 hours the loss in weight shall not exceed 2 per cent and the penetration of the residue at 80 degrees F. and for the period of 30 seconds using the same instrument as described above shall not be reduced more than 50 per cent.
6. The total soluble in carbon bisulphide shall not be less than 99 per cent.
7. The total soluble in 88 degree naptha shall not be less than 70 per cent.
8. The total inorganic matter or ash shall not exceed one per cent.
9. Cold Test.

a. A cube of the asphalt one inch on edge shall be soft and malleable at a temperature of zero degrees F.

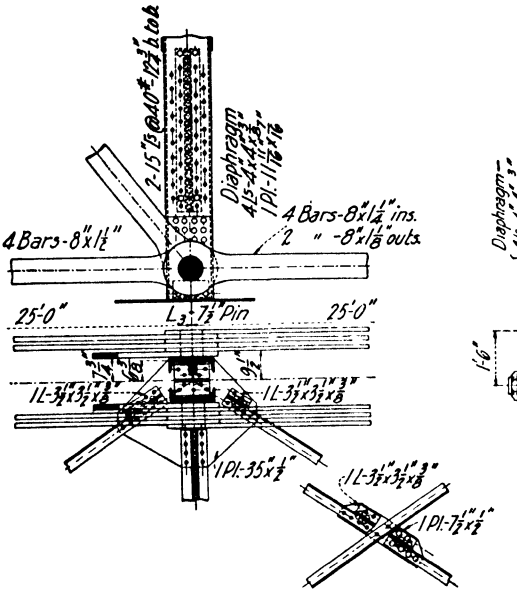


FIG. 27. FLOORBEAM CONNECTION. NORTHERN PACIFIC R. R.

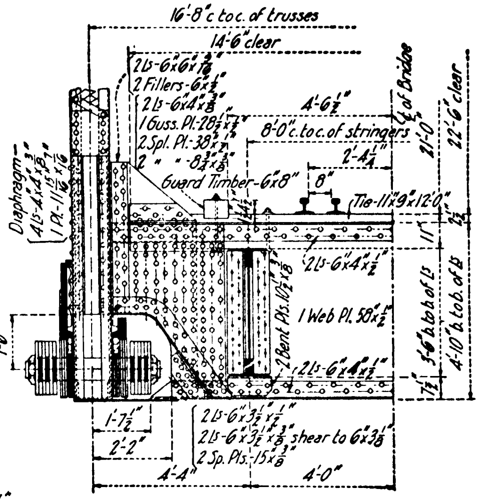


FIG. 28. FLOORBEAM CONNECTION. NORTHERN PACIFIC R. R.

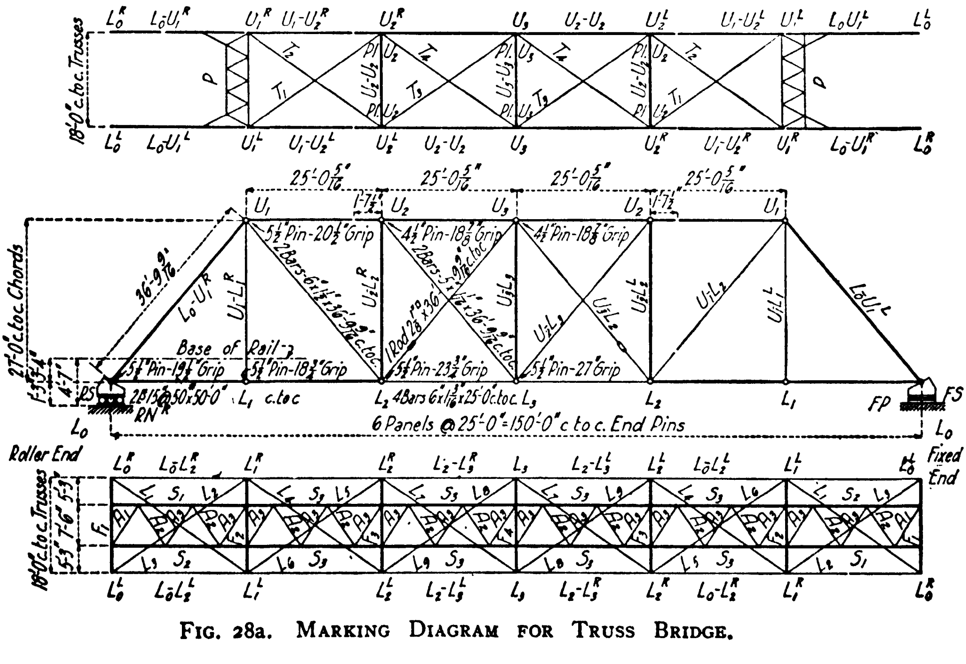


FIG. 28a. MARKING DIAGRAM FOR TRUSS BRIDGE.

b. A film of the asphalt having a thickness not less than $\frac{1}{16}$ inch shall be so pliable at zero degrees F. that it can be bent in a radius of 2 inches. The total time consumed in the bending of this film shall not exceed 3 seconds.

10. The asphalt shall not be affected by any of the following solutions, after being immersed in them for a period of 3 days:—(a) a 25 per cent solution of sulphuric acid; (b) a 25 per cent solution of hydrochloric acid; (c) a 20 per cent solution of ammonia.

FLOORBEAM CONNECTIONS.—The details of floorbeam connections depend upon the clearance, depth of truss, length of panels and type of floor. A standard type of floorbeam connection for a pin-connected truss of 150 ft. span is shown in Fig. 28, and details of the lower lateral connection are shown in Fig. 27. Details of a floorbeam connection for a pin-connected truss with

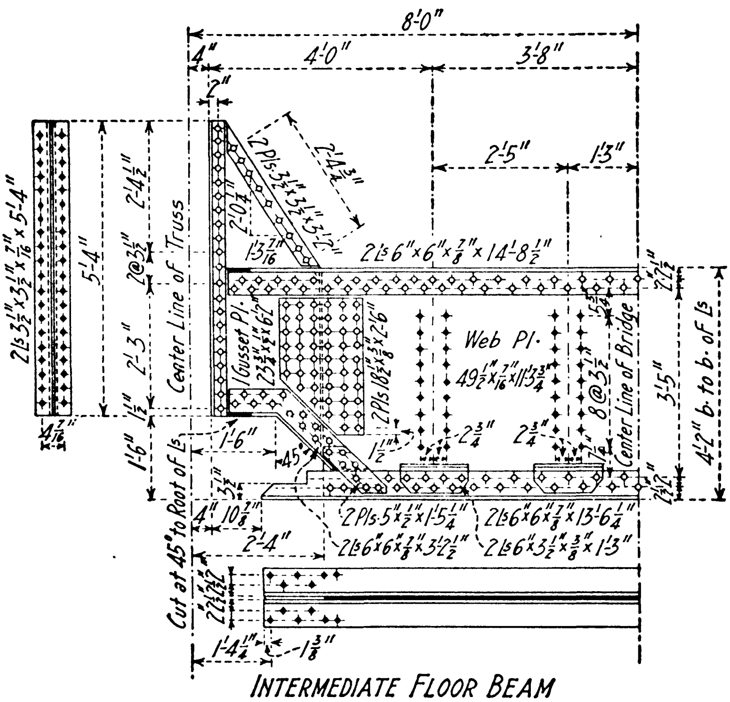


FIG. 29. INTERMEDIATE FLOORBEAM CONNECTION. A. T. & S. F. RY.

four stringers is shown in Fig. 29. Details of a floorbeam for a riveted truss bridge are shown in Fig. 40. Details of an end floorbeam are shown in Fig. 40. Details of the standard end floorbeam of the A. T. & S. F. Ry. are shown in Fig. 30. The end floorbeam in Fig. 30 is supported directly on the end pin, and gives a very satisfactory solution of a difficult problem and requires the driving of a minimum number of field rivets.

PEDESTALS AND SHOES.—Details of standard cast steel pedestals and shoes as designed by the Chicago, Milwaukee & St. Paul Ry. are shown in Fig. 31, Fig. 33, and Fig. 34. Details of segmental rollers are shown in Fig. 32, and Fig. 35. Details of expansion bearings for plate girders are shown in Fig. 36, and Fig. 37. Details of a built-up end shoe with circular rollers are shown in Fig. 40. Details of a built-up end shoe and segmental rollers are shown in Fig. 41.

EXAMPLES OF PLATE GIRDERS.—Details of an 85-ft. span single track deck railway plate girder bridge as designed for the Kansas City, Mexico & Orient R. R., by Mr. Ira G. Hedrick, Consulting Engineer, are shown in Fig. 36. The upper flanges are made of four angles

without cover plates, so that the ties may be of uniform thickness and there will be no rivet heads to interfere with placing the ties. The lower flanges are made of angles with cover plates. These plans represent the most modern practice in the design of deck plate girder railway bridges.

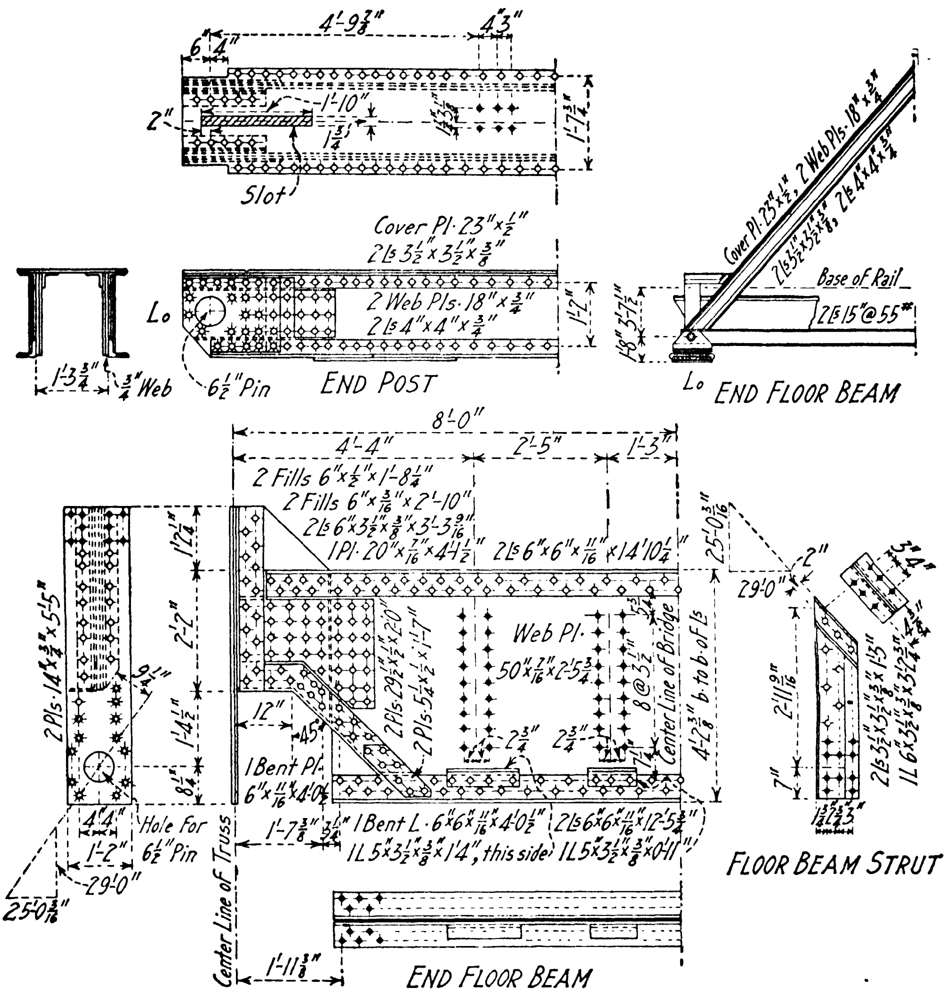


FIG. 30. END FLOORBEAM CONNECTION. A. T. & S. F. RY.

Details of a 60-ft. span single track through railway plate girder bridge as designed for the Harriman Lines are shown in Fig. 37. The details of the bearings are shown. Rollers are used for the expansion ends of spans of 75 ft. and over. Data on standard plate girder bridges designed under Common Standard Specifications 1006 are given in Table I.

EXAMPLES OF TRUSS BRIDGES.—The marking diagram for a truss railway bridge is shown in Fig. 28a. The lower chord joints are marked L_0, L_1, L_2, etc., while the upper chord joints are marked U_1, U_2, etc. In detailing a truss an inside view of the left end of the farther truss is shown; this is marked right as shown. Details of a single track through riveted truss

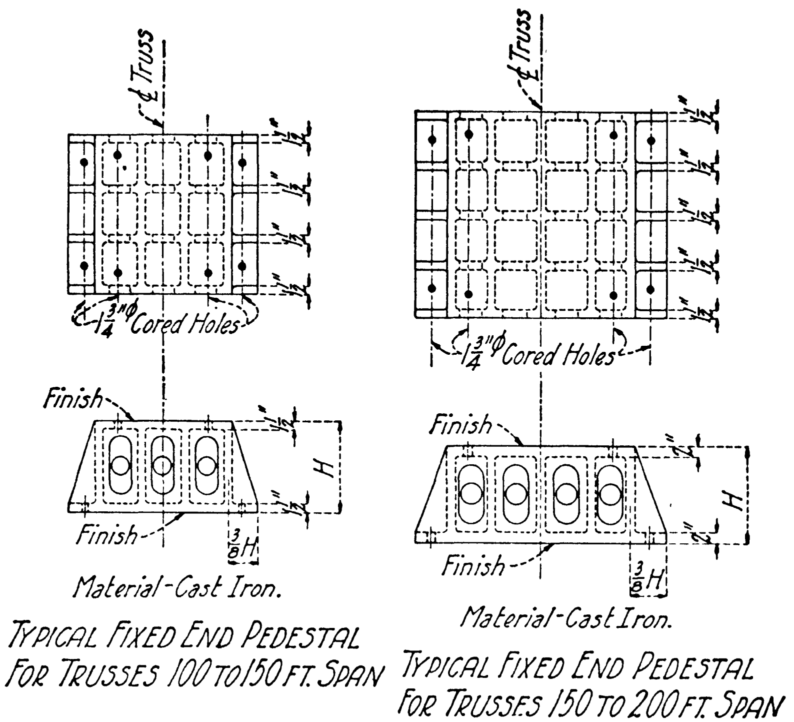


FIG. 31. PEDESTALS. CHICAGO, MILWAUKEE & ST. PAUL RY.

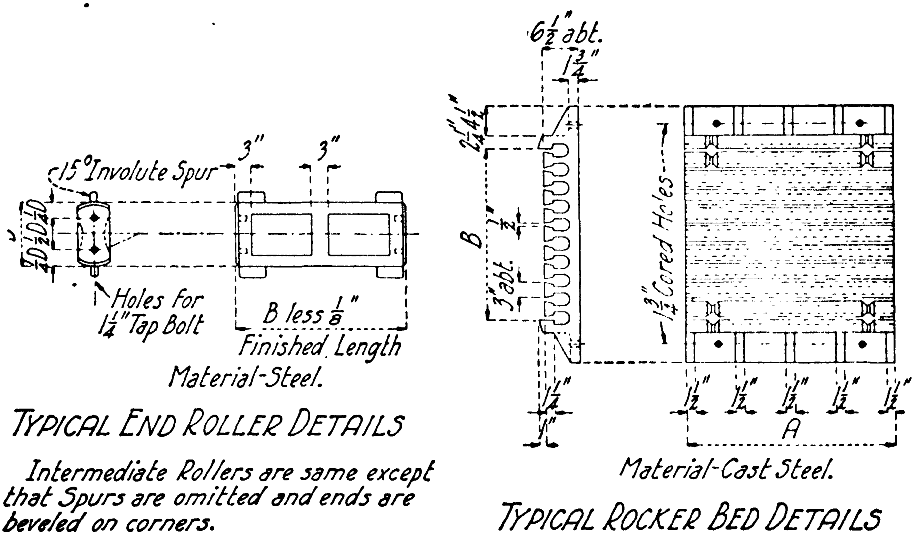


FIG. 32. ROLLERS. CHICAGO, MILWAUKEE & ST. PAUL RY.

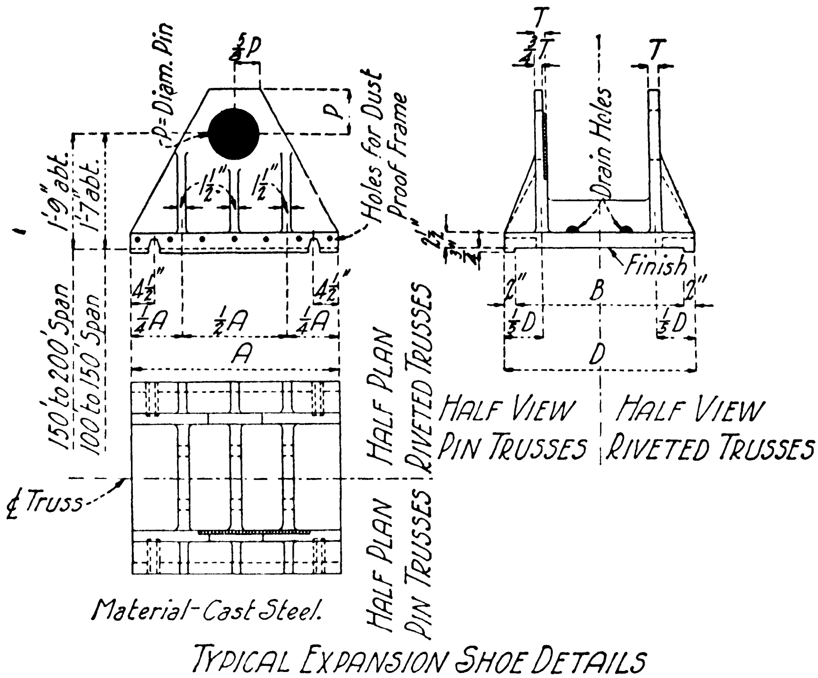


FIG. 33. SHOE DETAILS. CHICAGO, MILWAUKEE & ST. PAUL RY.

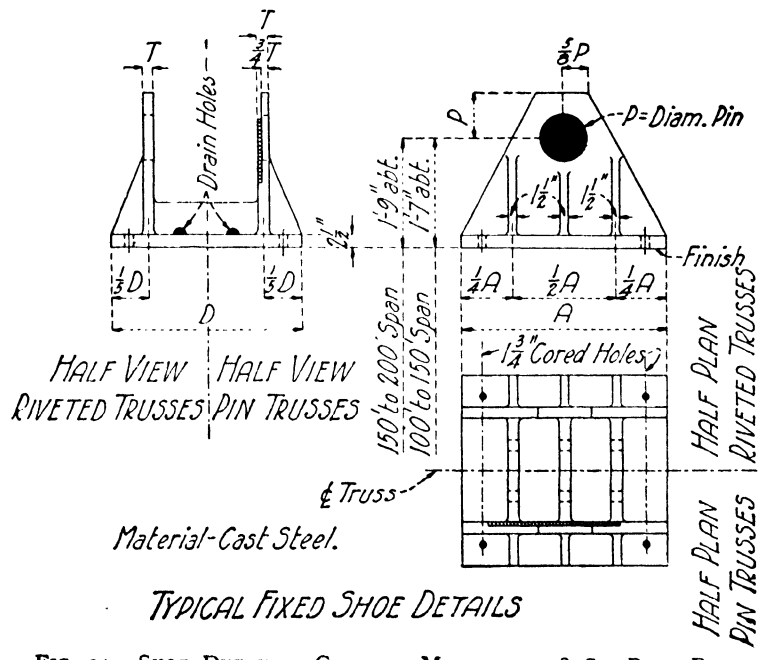


FIG. 34. SHOE DETAILS. CHICAGO, MILWAUKEE & ST. PAUL RY.

bridge designed for the Kansas City, Mexico & Orient R. R., by Mr. Ira G. Hedrick, Consulting Engineer, are shown in Fig. 39 and Fig. 40. The end-posts and top chords are made of two 15 inch channels with a cover plate, and the lower chords, the posts and the main ties are made of two channels with the flanges turned in. The total weight of the steel in the span was 303,000 lb.

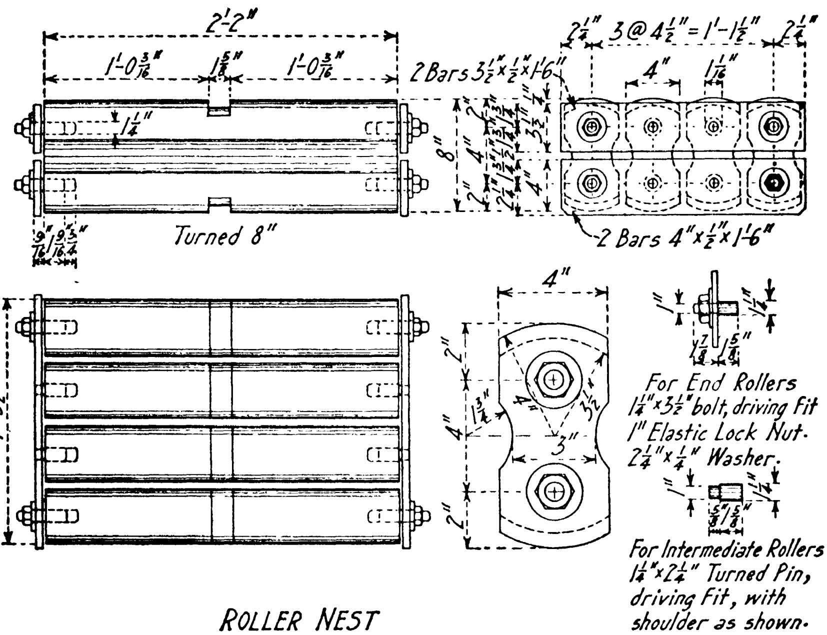


FIG. 35. DETAILS OF SEGMENTAL ROLLERS FOR GIRDERS.
CHICAGO, MILWAUKEE & ST. PAUL RY.

Details of a double track through riveted truss bridge designed for the Chicago & Northwestern Ry. are given in Fig. 41. The bridge has a span of 170 ft., the trusses are spaced 29 ft. 1 in. centers, and the bridge has a vertical clearance of 22 ft. 6 in. This bridge has trusses with triple intersection webs, and has a ballasted track carried on a steel plate trough floor. This bridge was designed for a dead load of 4,570 lb. per lineal foot for each truss and an E 50 live load. There is a top lateral system of multiple X-bracing made with pairs of angles latticed, and sway bracing of transverse top chord struts and portals.

Detail shop drawings of the end-post of a pin-connected truss bridge are given in Fig. 8, Chapter XII, and the detail shop drawings of the end section of the top section of the top chord of the same bridge are given in Fig. 9, Chapter XII.

Details of a single track pin-connected truss bridge designed by Mr. Ralph Modjeski for the Northern Pacific R. R. are given in Fig. 44, Fig. 45 and Fig. 46.

SPECIFICATIONS FOR RAILWAY BRIDGES.*—To determine the present practice in the design of railway bridges the author has made a study of the latest available specifications. As a basis for comparison the sixteen specifications given in Table XI, were selected as being representative of the best practice. Several other prominent railroads have adopted the specifications of the American Railway Engineering Association, so that the sixteen specifications cover the major part of the railroad mileage in North America. The standard specifications of the Chicago, Milwaukee and St. Paul Ry., the New York, New Haven and Hartford R. R., and the Canadian Society of Civil Engineers, all adopted in 1912, are based on the standard speci-

* For review of railway bridge specifications in 1924, see latter part of this chapter.

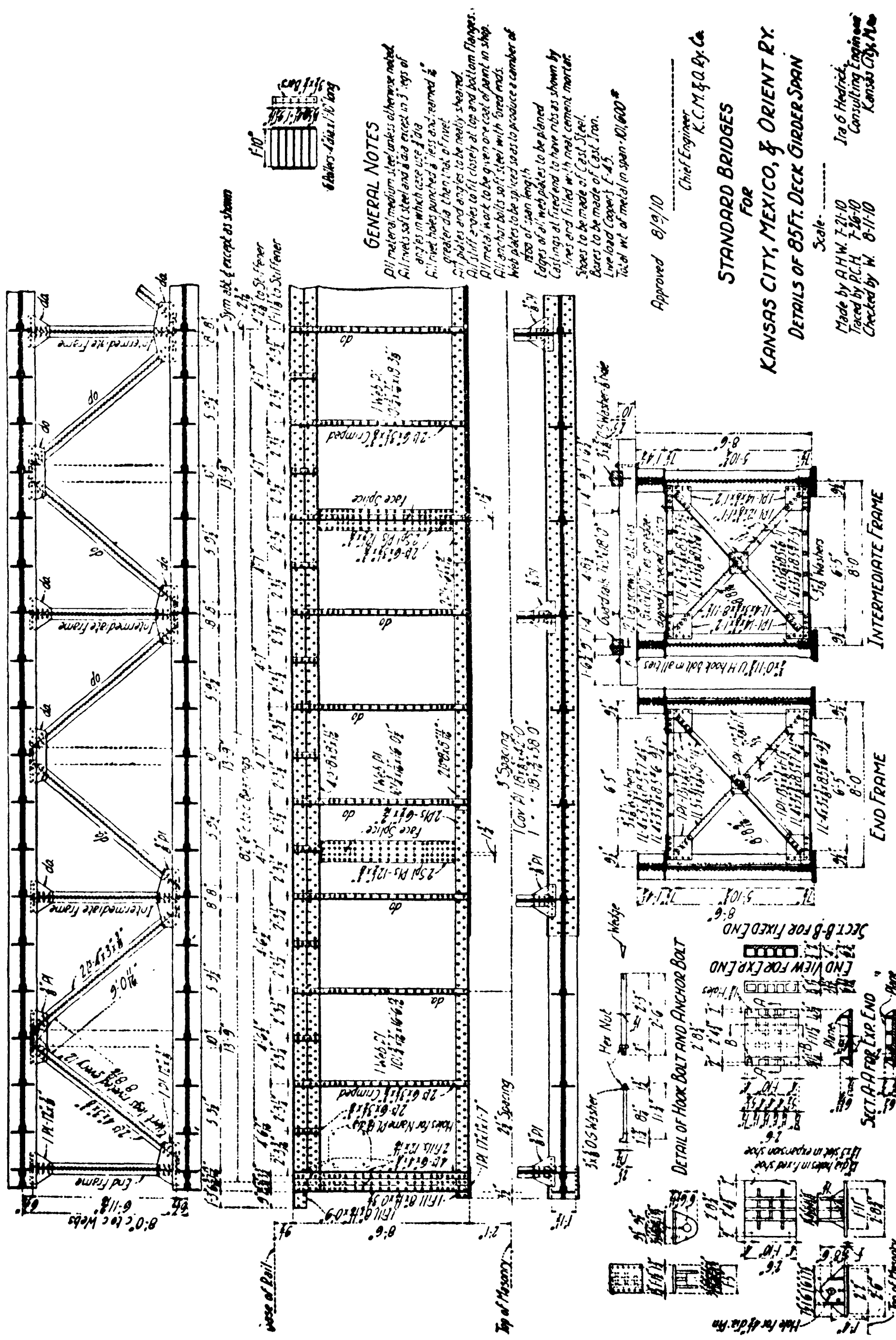

GENERAL NOTES
STANDARD BRIDGES
FOR
KANSAS CITY, MEXICO, & ORIENT RY.
DETAILS OF 85 FT. DECK GIRDER SPAN
INTERMEDIATE FRAME
END FRAME
DETAIL OF HOOK BOLT AND ANCHOR BOLT

FIG. 36.

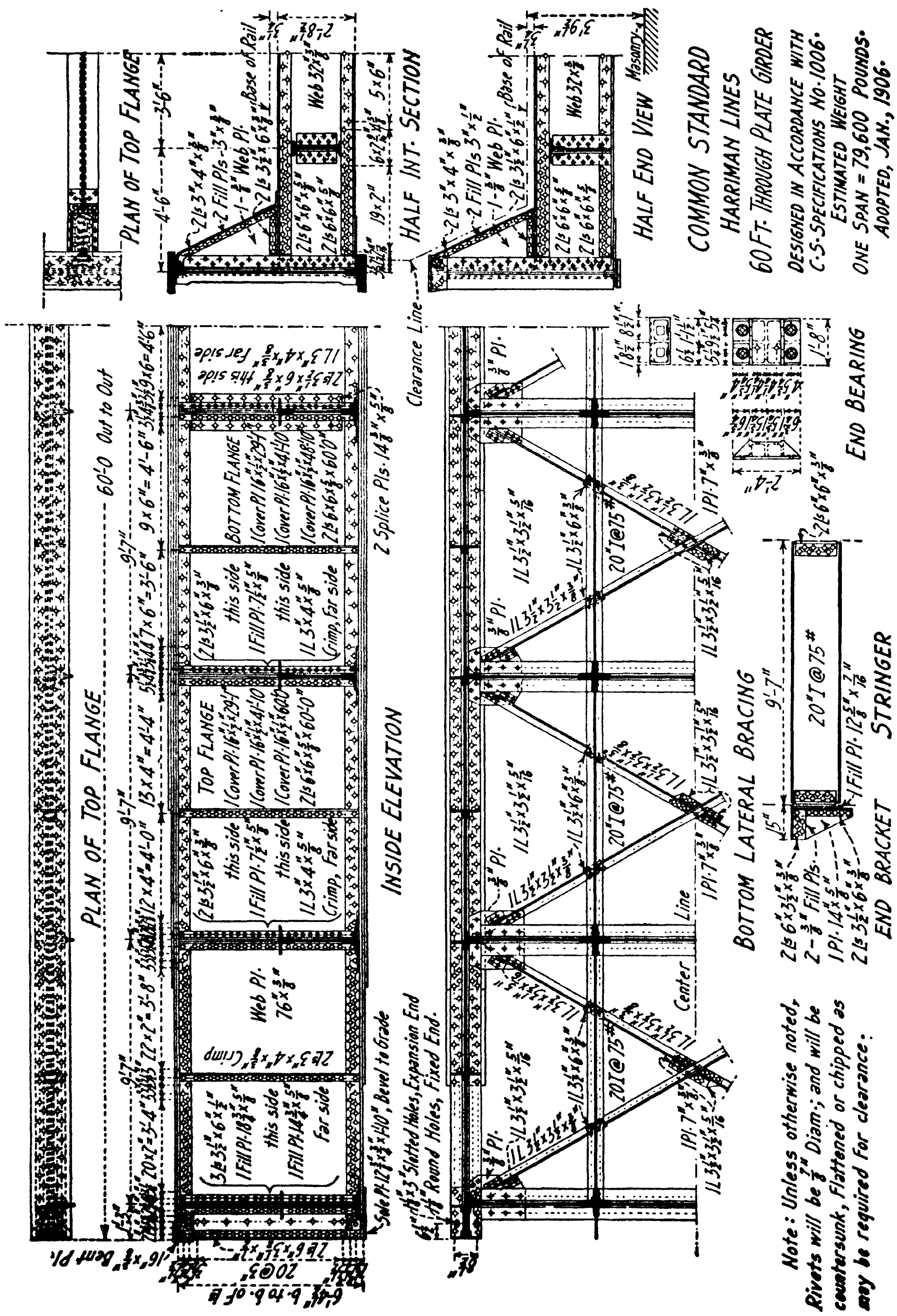


FIG. 37. THROUGH PLATE GIRDER BRIDGE.

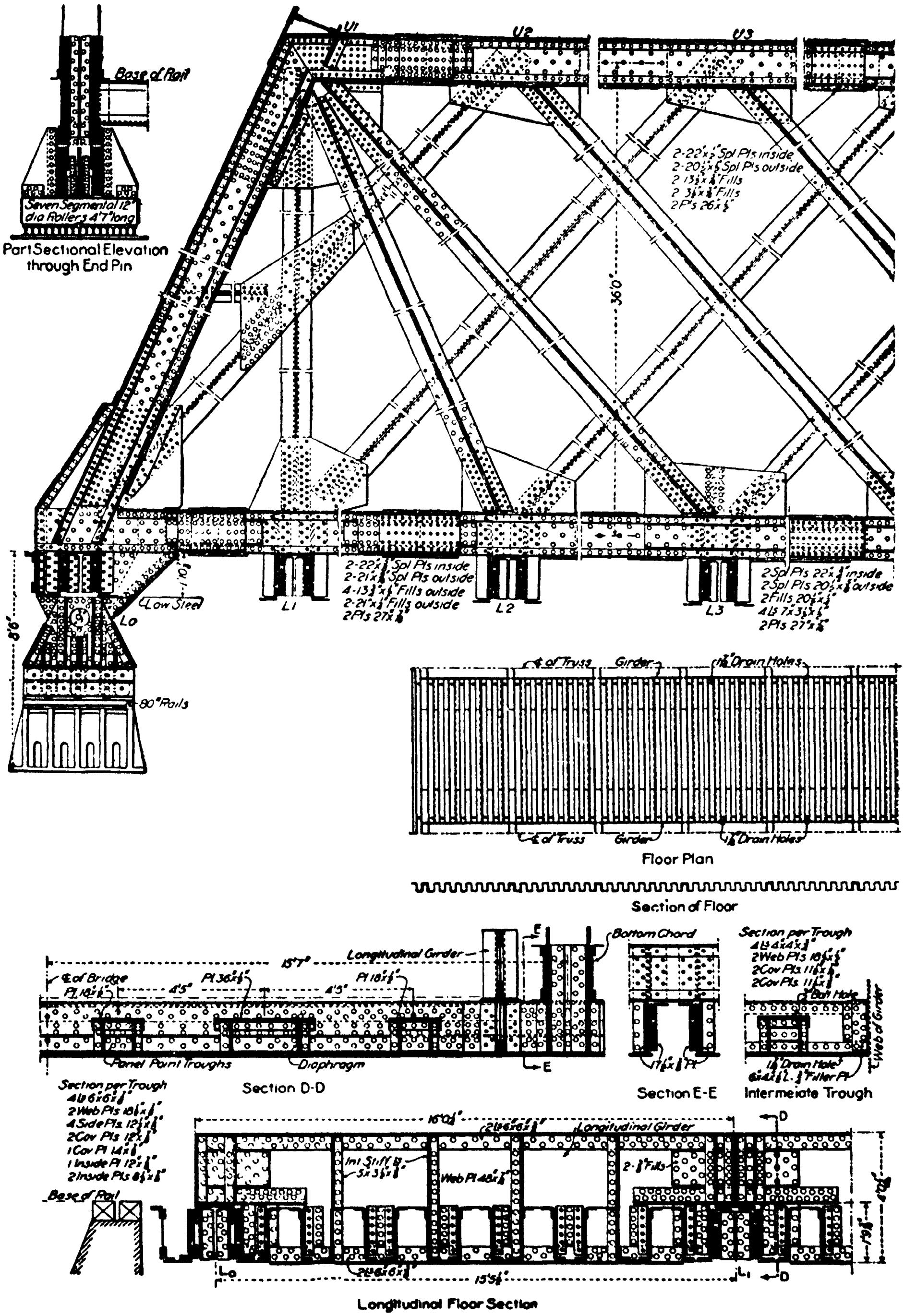


FIG. 38. CHICAGO & NORTHWESTERN RAILWAY BRIDGE.

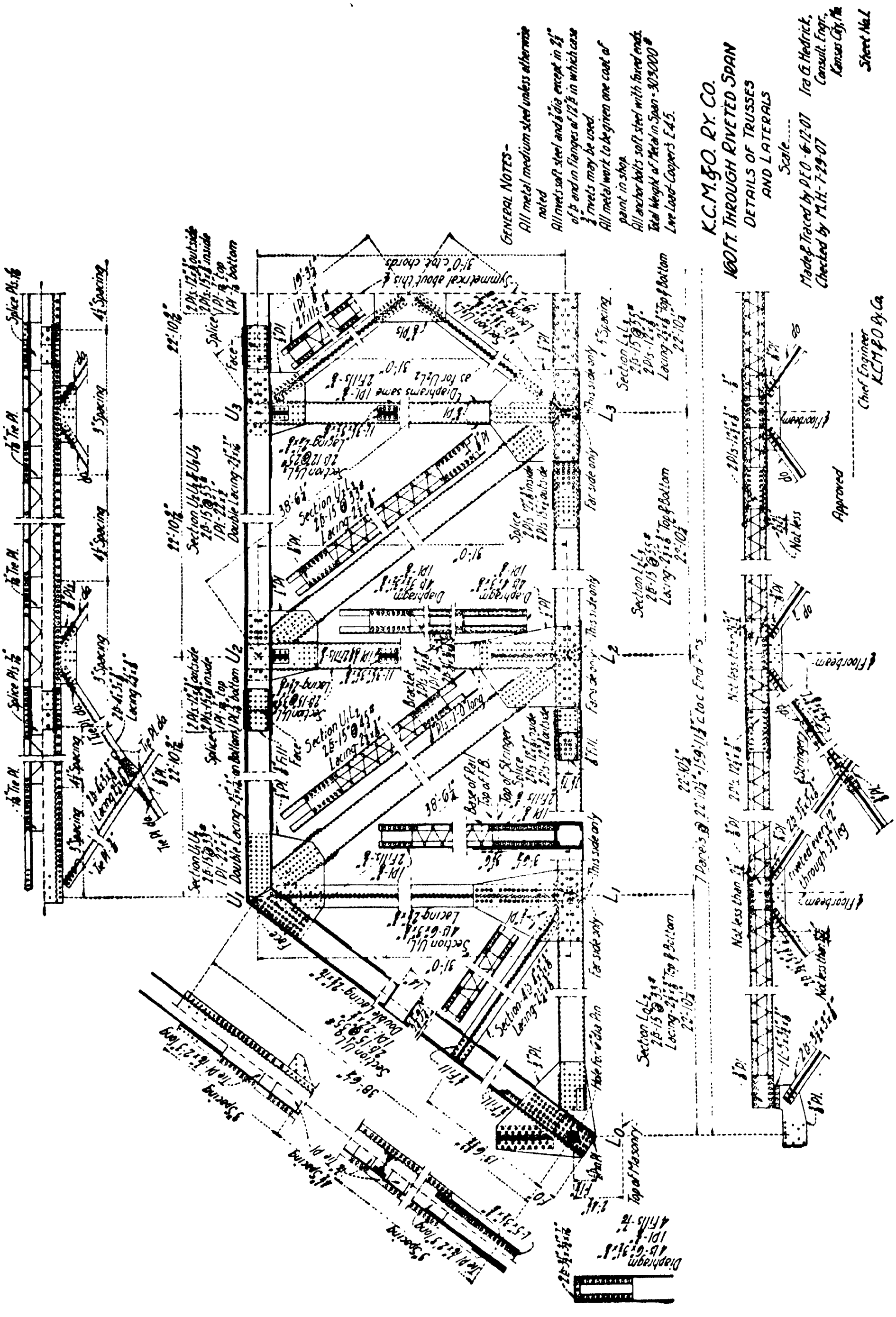


FIG. 39. SINGLE TRACK THROUGH RIVETED TRUSS RAILWAY BRIDGE.

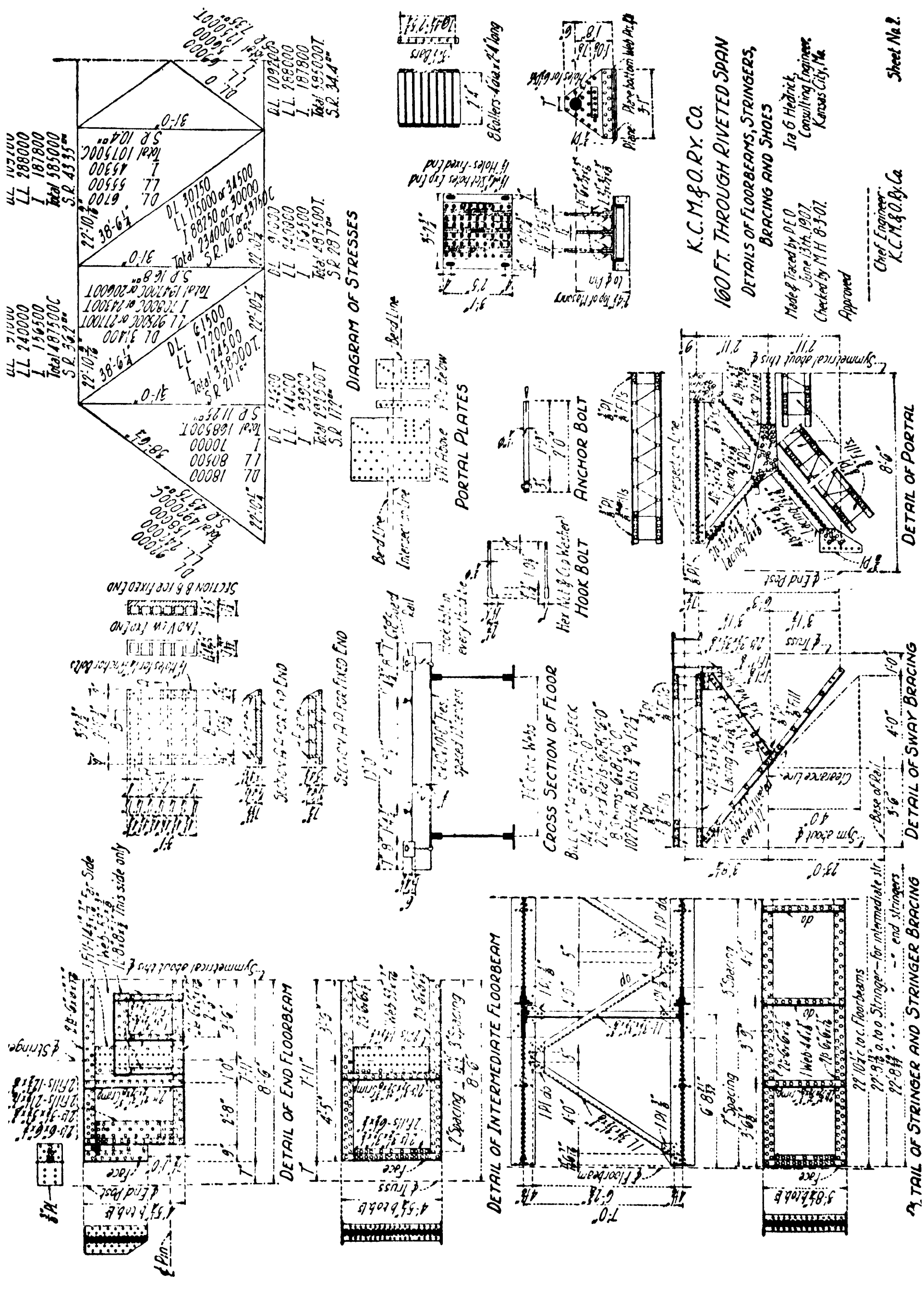


FIG. 40. SINGLE TRACK THROUGH RIVETED TRUSS RAILWAY BRIDGE.

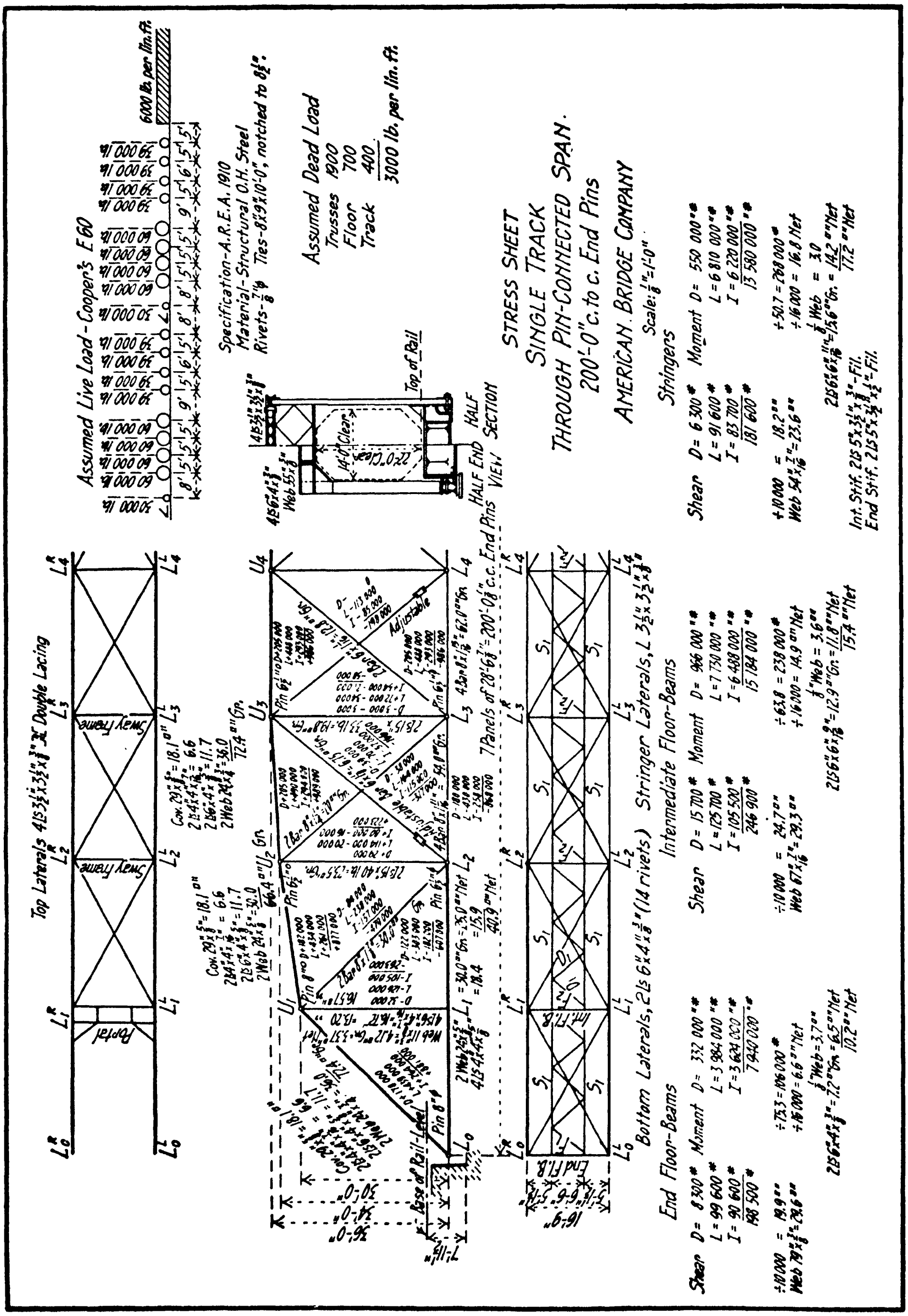


FIG. 41. DESIGN FOR 200 FT. SPAN TRUSS RAILWAY BRIDGE.

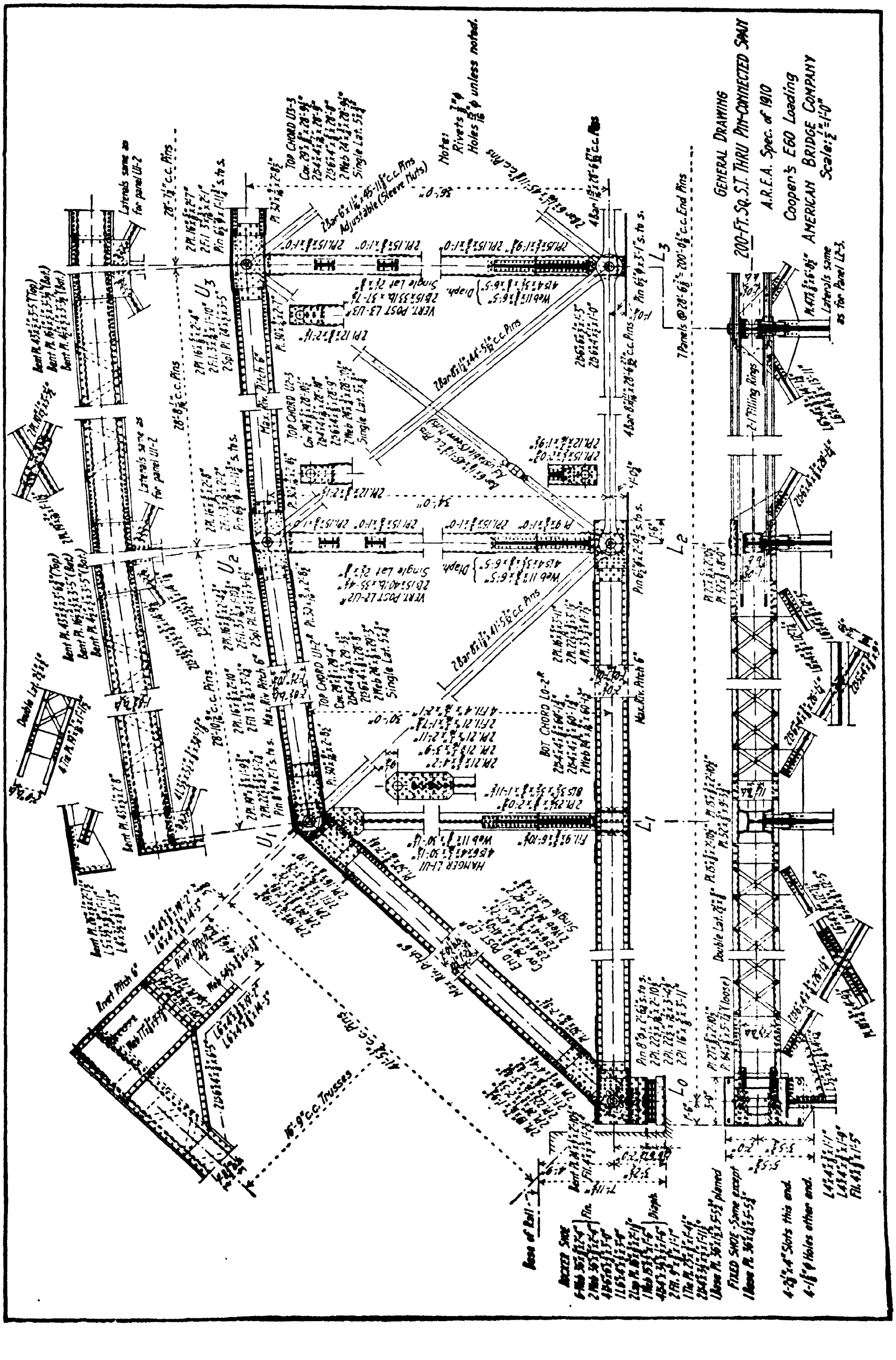


FIG. 42. DESIGN FOR 200 FT. SPAN TRUSS RAILWAY BRIDGE.

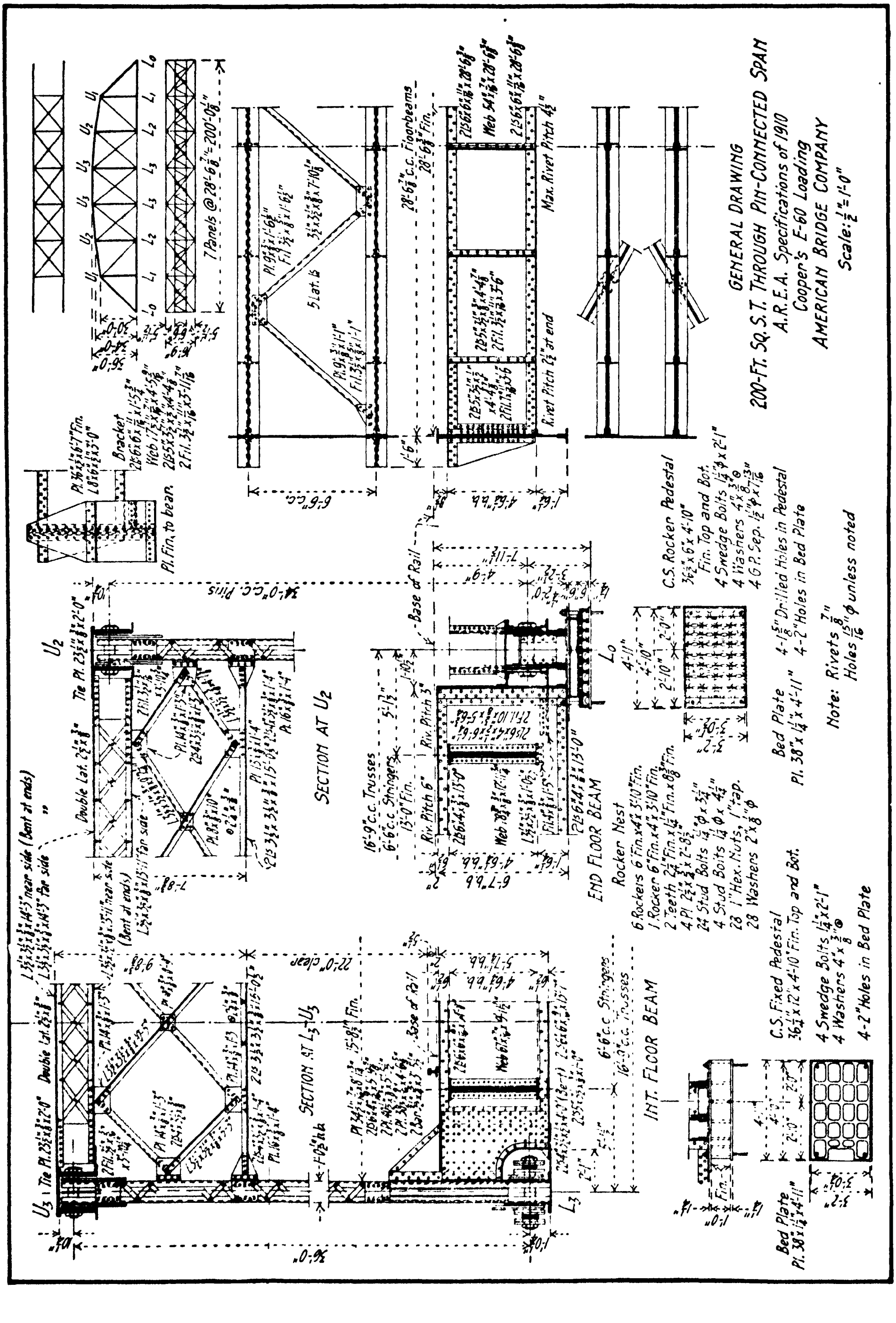


FIG. 43. DESIGN FOR A 200 FT. SPAN TRUSS RAILWAY BRIDGE.

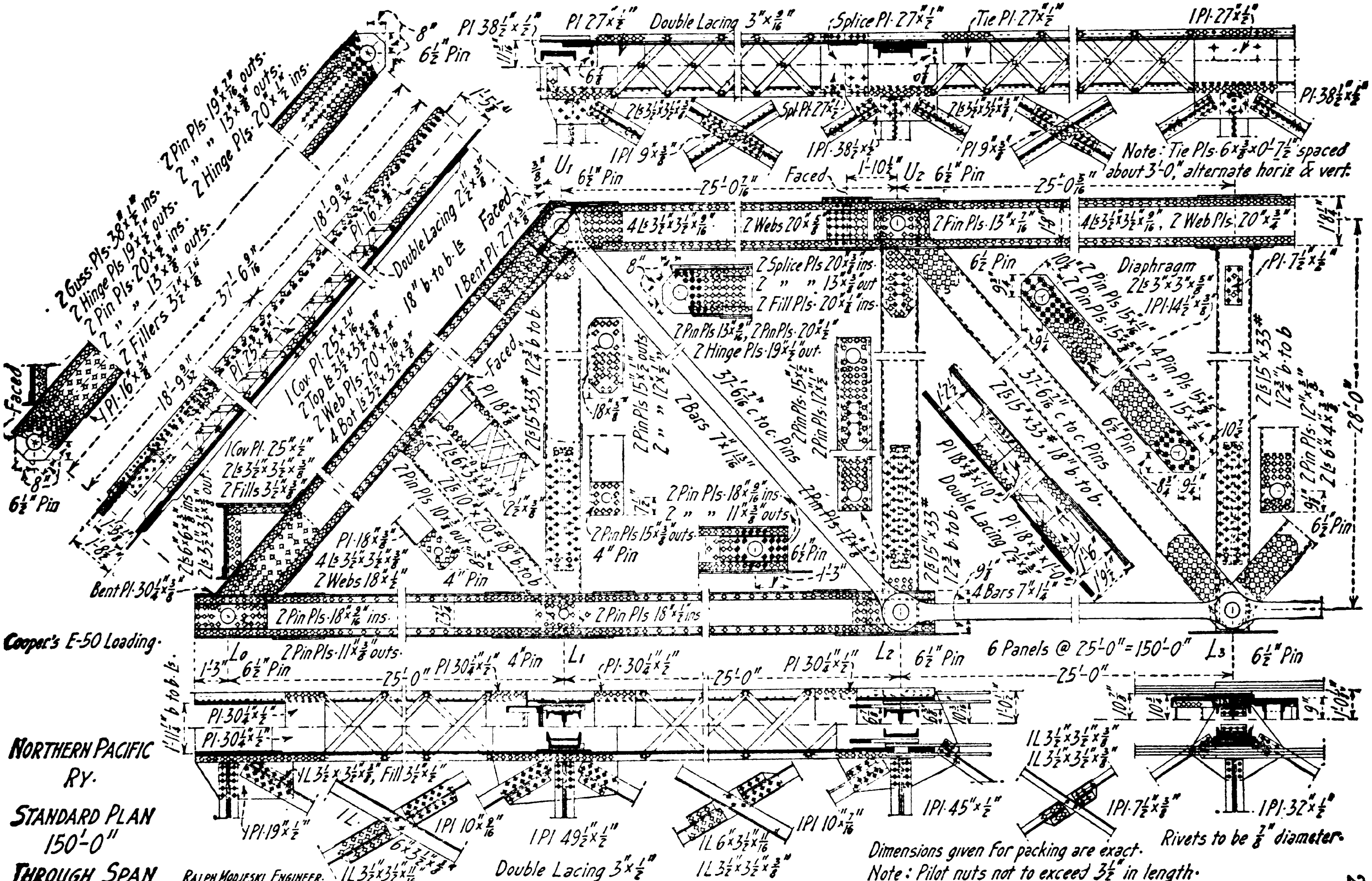


FIG. 44. SINGLE TRACK THROUGH PIN-CONNECTED RAILWAY BRIDGE.

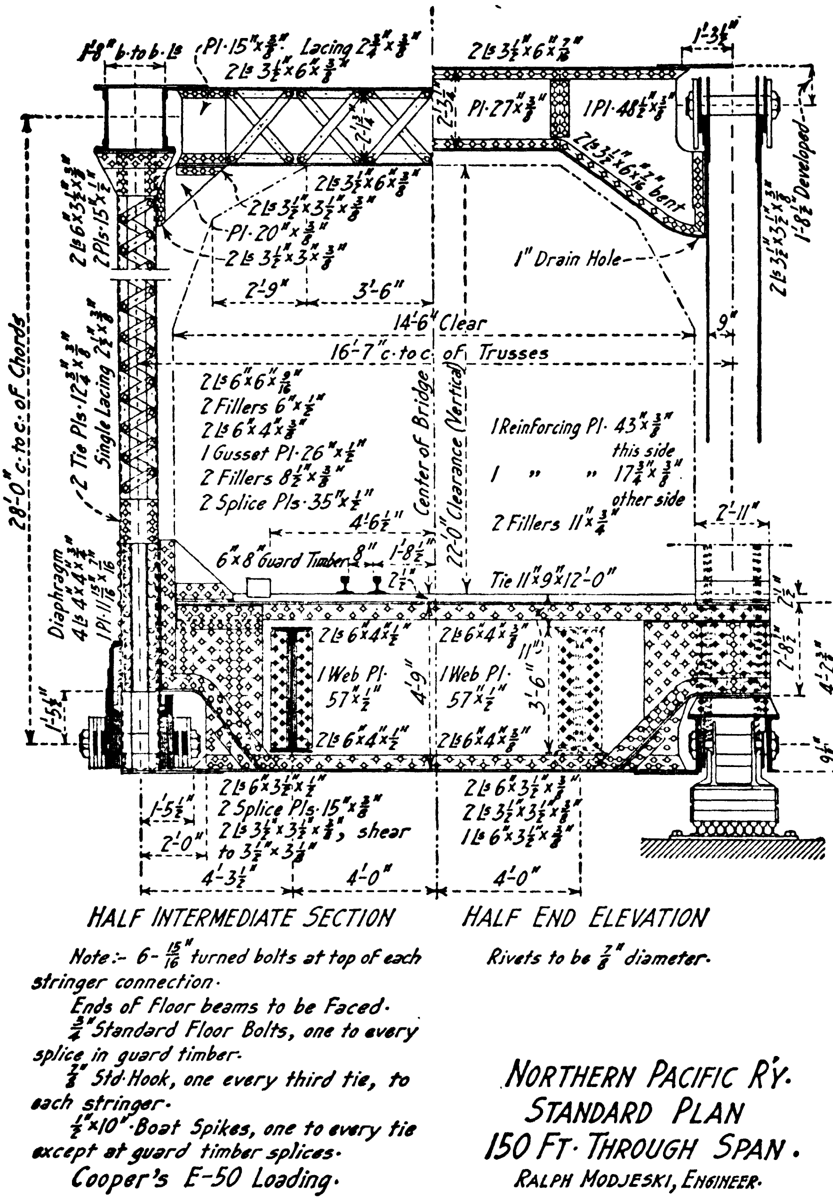
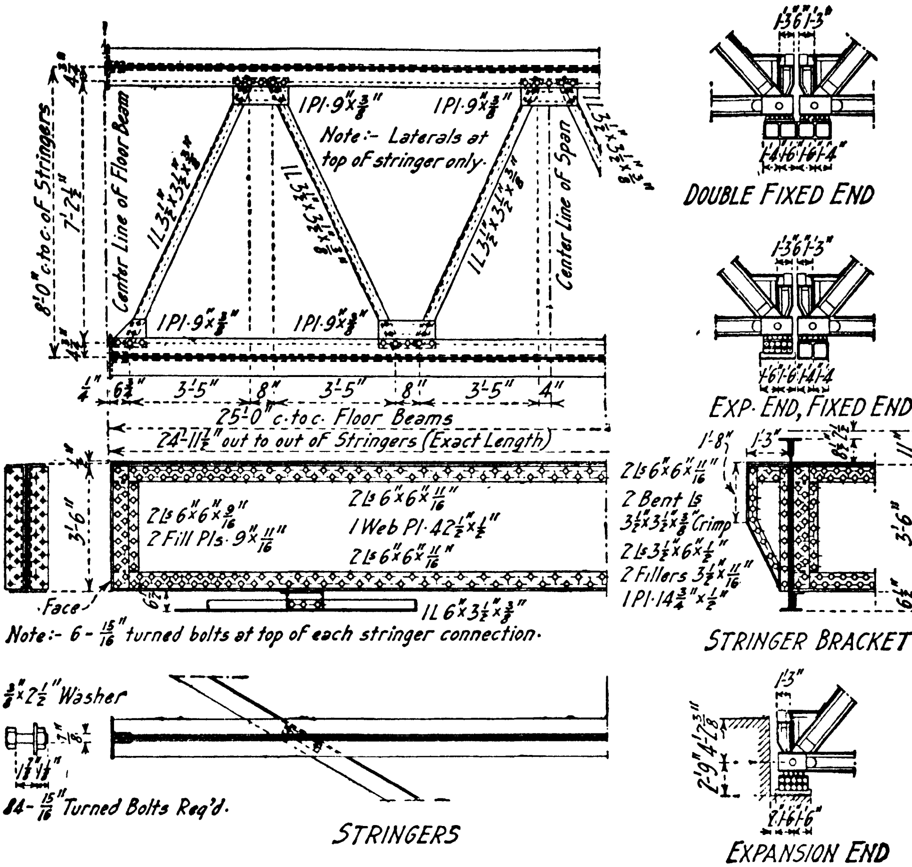


FIG. 45. SINGLE TRACK THROUGH PIN-CONNECTED RAILWAY BRIDGE.



BILL OF TRACK MATERIAL
FOR ONE SPAN

Material	No. Req'd	Size	Length	Mark
Cross Ties	122	9"x11"	12'-0"	
Guard Timber	18	6"x8"	20'-0"	
Std. Floor Bolt	16	3/4" dia.	1'-5 1/4"	B·⌑
Hook Bolts	88	7/8" dia.	1'-4 1/4"	C·⌑
Boat Spikes	228	1/2" dia.	0'-10"	

Cooper's E-50 Loading.

Note:- Holes to be punched with 3/4" diameter die, and reamed to 15/16" diameter after assembling.
Rivets to be 7/8" diameter.
Total weight of one span including track bolts and bearings = 315,490 lbs.

NORTHERN PACIFIC RY.
STANDARD PLAN
150 FT. THROUGH SPAN.
RALPH MODJESKI, ENGINEER.

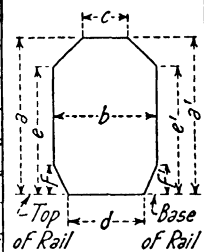
FIG. 46. SINGLE TRACK THROUGH PIN-CONNECTED RAILWAY BRIDGE.

fications of the American Railway Engineering Association; the specifications in each case differing from the specifications of the American Railway Engineering Association only in requirements for clearances, and in minor clauses, and clauses required to cover individual practice, and local conditions of the individual roads.

TABLE XI.

RAILWAY BRIDGE CLEARANCES

Specification	a	a'	b	c	d	e	e'	f	f'
1·American Ry·Eng·Assoc·, 1910	22'0"		14'0"	6'0"	10'6"	18'0"		4'0"	
2·A·T·& S·F·Ry·System, 1902		23'6"	14'0"	7'0"	10'0"		19'0"		4'0"
3·Baltimore & Ohio, 1904		22'0"	14'0"	6'0"	10'6"		18'0"		4'0"
4·Boston & Maine (In Canada), 1912	22'0"		16'0"	8'0"	13'0"	19'0"		4'0"	
5·Chi·Mil·& St·P·R·R·, 1912	23'0"		15'0"	7'0"	11'0"	19'0"		2'6"	
6·Chi·Rock Island & Pac R.R., 1906	23'6"		14'0"	7'0"	11'0"	18'6"		4'0"	
7·Common Standard, 1909		24'0"	15'0"	6'0"	11'0"		19'0"		4'0"
8·Cooper, 1906		21'0"	14'0"				15'0"		2'0"
9·Illinois Central, 1911	22'0"		16'0"	8'0"	11'0"	18'0"		4'0"	
10·Kan·City, Mexico & Orient, 1907		23'0"	15'0"	7'0"	11'0"		19'0"		4'6"
11·Lehigh Valley, 1911	22'0"		14'0"	6'0"	11'0"	18'0"		4'0"	
12·New York Central 1910	22'0"		15'0"	8'0"	11'0"	15'0"		4'0"	
13 New York, New Haven & Hartford, (In Canada) 1912	22'0"		16'0"	8'0"	13'0"	18'0"		4'0"	
14·Penna·Lines West of Pittsburgh, 1906		21'6"	14'0"	6'0"	10'0"		16'0"		4'0"
15·National Lines of Mexico, 1907		22'0"	15'0"	6'0"	11'0"		16'0"		4'0"
16·Canadian Society Civil Engineers, 1912		22'6"	16'0"	7'0"	10'6"		17'6"		3'3"



For Double Track add distance c·to c· of tracks to above Figures b, c, and d·

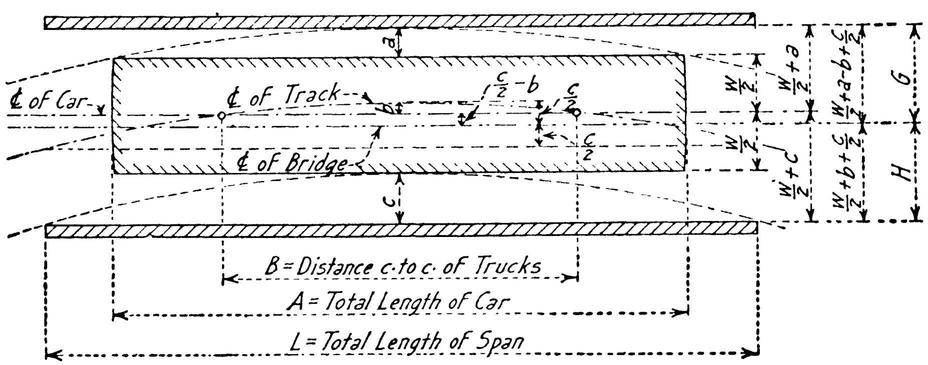


FIG. 47. CLEARANCE DIAGRAM.

The present practice is to use the specifications of the American Railway Engineering Association as a basis for specifications and to add such additional clauses as may be necessary to cover the practice of the individual railroad. Several railroads have adopted the specifications of the American Railway Engineering Association and issue supplementary instructions to cover their individual practice; see standards of Chicago, Milwaukee & St. Paul Ry. which follow the A. R. E. A. specifications in this chapter. The specifications of the American Railway Engineering

Association are reprinted in the last part of this chapter. To show the present practice in the design of railway bridges as given in the sixteen different specifications the most important variations from the American Railway Engineering Association Specifications will be briefly discussed. The sections in the specifications of the American Railway Engineering Association will be referred to by number.

§2. **Clearances.**—The clearances for through single track bridges on tangent are given in Table XI. The clearances on curves differ considerably. Standard formulas for calculating bridge clearances on curves are as follows:

Nomenclature, Fig. 47:—

D = degree of curve
R = radius of curve, in feet
w = clearance width on tangent
a = mid-ordinate to chord of length A
b = mid-ordinate to chord of length B
c = mid-ordinate to chord of length L
e = amount of superelevation in feet which is taken up *in floor of span*
h = height of car or distance from top of upper flange or chord, whichever is least
s = additional clearance required on account of superelevation
G = outside clearance from center line of bridge
H = inside clearance from center line of bridge

Formulas:—

$$a = \frac{A^2}{8R} \text{ (nearly)} = \frac{A^2 \cdot D}{8 \times 5730} = .000021817A^2 \cdot D$$

$$b = .000021817B^2 \cdot D$$

$$c = .000021817L^2 \cdot D$$

$$s = \frac{e}{5} \times h = 0.2e \cdot h \text{ (c. to c. rails} = 5 \text{ ft. nearly)}$$

$$G = \frac{w}{2} + a - b + \frac{c}{2}$$

$$H = \frac{w}{2} + b + \frac{c}{2} + s$$

For Standard Car

$A = 80'\text{-}0''$ $\qquad B = 60'\text{-}0''$

$a = 0.1396D$

$b = .07854D$

$$G = \frac{w}{2} + (.06109 + .000010909L^2)D$$

$$H = \frac{w}{2} + (.07854 + .000010909L^2)D + 0.2e \cdot h$$

The following specifications indicate the present practice of several railroads.

New York Central Lines.—Single-track through bridges on curves shall have the location of the trusses or girders and the width between clearance lines as shown in Figs. 48 and 49.

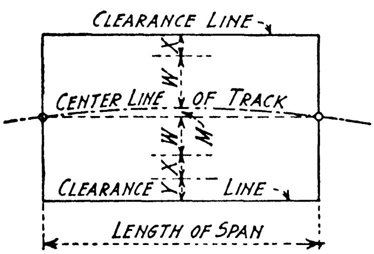


FIG. 48.

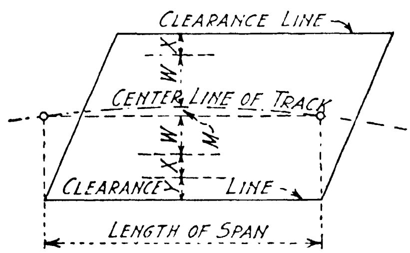


FIG. 49.

W = lateral clearance from center line of track required by clearance diagram for tangent alinement.
M = middle ordinate of curve for a chord equal to span length.
X = addition for overhang of a car 85 ft. long, with trucks 60 ft. c. to c.; to be taken as one inch for each degree of curve.
Y = addition in inches (on the inside of the curve only) on account of the superelevation of the outer rail, to be taken as follows:
For heights from 15 ft. to 22 ft. above the top of rail; $Y = 3$ inches per inch of superelevation.
For heights from 3 ft. 9 in. to 15 ft. above top of rail; $Y = s \cdot h/5$ (to use with $W = 7$ ft. 6 in.).
For heights from top of rail to 3 ft. 9 in. above; $Y = s(h + 1.5)/4$.
s = superelevation in inches.
h = height above top of rail in feet

Cooper's Specifications.—The additional clearance for curves is to be as follows: $0.85D$ = inches on each side; $1.70D$ = inches between track; where D = degree of curve.

N. Y., N. H. & H. R. R.—The additional clearances on curves will be as follows: $1.00 \times D$ = inches on each side; $1.75D$ = inches between tracks, where D = degree of curve.

Types of Bridges.—The present practice is to use plate girders for spans up to 110 or 120 ft., riveted trusses for spans of from 100 to 200 or 250 ft., and pin-connected trusses for spans of about 200 ft. and upwards. Riveted truss bridges of 300 and 400 ft. span are not uncommon. The types of bridges and minimum lengths of span as given in twelve specifications are given in Table XII.

TABLE XII.

TYPES OF BRIDGES AND LENGTHS OF SPAN.

Specification	Rolled Beams Ft.	Plate Girders Ft.	Riveted Trusses Ft.	Pin Connected Trusses Ft.
2. A. T. & S. F. Ry. System, 1902	26 to 34	26 to 106	106 to 150	150 and up
3. Baltimore & Ohio, 1904	30	30 to 110	100 to 150	150 and up
6. Chi., Rock Island & Pac. R. R., 1906	19	19 to 110	100 to 200	200 and up
7. Common Standard, 1909	19	19 to 100	100 to 150	150 and up
8. Cooper, 1906	20	20 to 120	120 to 150	150 and up
9. Illinois Central, 1911	21	21 to 100	100 to 150	150 and up
10. Kansas City, Mexico & Orient, 1907	20	20 to 100	100 to 250	250 and up
11. Lehigh Valley, 1911	25	25 to 110	110 to 160	160 and up
12. New York Central, 1910	25	25 to 110	110 to 180	180 and up
14. Penna. Lines West of Pittsburgh, 1906		to 100	100 to 250	250 and up
15. National Lines of Mexico, 1907	30	25 to 80	80 to 150	150 and up
17. Department of Railways of Canada, 1908	18	18 to 100	100 to 200	200 to 600

§3. **Spacing of Trusses.**—The present practice is not to put requirements for spacing of trusses, lengths of span, types of bridge, etc., in the specifications but to prepare office standards for the use of engineers and draftsmen. Data on spacing stringers, girders and trusses are given in Table XIII. The spacings for Illinois Central R. R. deck girders are given in Figs. 11, 12 and 13, and of Common Standard Bridges in Table I.

The Chicago, Milwaukee and St. Paul Ry. spaces girders 7 ft. 6 in. west of the Missouri River, and 8 ft. east of the Missouri River. The Northern Pacific R. R. spaces stringers 8 ft. for spans of 150 to 200 ft.; and deck girders 8 ft. for 80 ft. spans.

§5. **Ties.**—The present practice is to calculate the size of stringers for the specified fiber stress. Fifteen specifications require that the wheel load be considered as carried by three ties, and one specification by four ties. Data on ties are given in Table XIV.

The Illinois Central R. R. uses ties on deck girders as follows:

Deck Spans.	Distance Centers.	Ties.
60 ft. and under	7 ft.	8 in. × 8 in. × 10 ft.
60 ft. to 80 ft.	8 ft.	8 in. × 10 in. × 12 ft.
80 ft. to 100 ft.	9 ft.	10 in. × 10 in. × 12 ft.
100 ft to 110 ft.	9½ ft.	10 in. × 12 in. × 12 ft.

§6. **Dead Loads.**—Data on dead loads are given in Table XV.

TABLE XIII.

SPACING OF GIRDERS AND TRUSSES

Specification	Girders		Trusses	
	Stringers	Deck Girders	Deck	Through
1·American Ry Eng·Assoc., 1910		Span/20	Span/20	Span/20
3·Baltimore & Ohio 1904	6'-6"	not less than 6'-6"	not less than 10'-0"	Span/20
6·Chicago, Rock Island & Pac. R·R; 1906	7'-0"	up to 60 Ft., 7'-0" 60 Ft. to 80 Ft., 8'-0"		Span/20
7·Common Standard, 1909	7'-0"	up to 60 Ft., 7'-0" 60 Ft. to 80 Ft., 8'-0" 80 Ft. to 100 Ft., 10'-0"	100 Ft. to 110 Ft., 10'-0" 110 Ft. to 130 Ft., 12'-0" 130 Ft. to 150 Ft., 14'-0"	
8· Cooper, 1906	6'-6"	not less than 6'-6"		
9· Illinois Central 1911	4 stringers spaced 2'-6"	up to 60 Ft., 7'-0" 60 Ft. to 80 Ft., 8'-0" 80 Ft. to 100 Ft., 9'-0" 100 Ft. to 110 Ft., 9'-6"	100 Ft. to 110 Ft., 10'-0" 110 Ft. to 130 Ft., 12'-0" 130 Ft. to 150 Ft., 14'-0"	
10·Kans. City, Mexico & Orient, 1907	7'-0"	up to 80 Ft., 7'-0" over 80 Ft., 8'-0"		Span/20
11·Lehigh Valley, 1911	6'-6"	up to 75 Ft., 6'-6" 75 Ft. to 100 Ft., 7'-0" 100 Ft. to 125 Ft., 7'-6" to 8'0		
12·New York Central, 1910	6'-6"	up to 75 Ft., 6'-6" over 75 Ft., 7'-6"		Span/15
14·Penna. Lines West of Pittsburgh, 1906	6'-6", for 4 stringers outer pair 7'-0", inner pair 3'-0"	6'-6"		
17·Department of Railways of Canada, 1908	8'-0"	Single Track, 8'-0" Double Track, 6'-6"	10'-0" or $\frac{1}{15}$ Span.	Span/20

§7. **Live Loads.**—Data for live loads are given in Table XVI. The type of engine is given in the second column and the weight in thousands of pounds of a single engine without tender is given in the third column; the special loadings and the spacing of the loads are given in the fourth and fifth columns; the impact formulas are given in the sixth column; the allowable tensile stresses are given in the seventh column, and the equivalent loading is given in the last column. The equivalent loading is found by multiplying the loading in the second column by 16,000 and dividing by the allowable tensile strength. The present standard loading on trunk lines is Cooper's E 60 loading.

The C. M. & St. P. Ry. uses E 60 followed by a train load of 7,000 lb. per lineal foot of track on ore roads; while the Duluth & Iron Range R. R. uses E 60 followed by a train load of 8,000 lb. per lineal foot of track.

In a paper entitled "Rolling Loads on Bridges" published in Bulletin No. 161, Am. Ry. Eng. Assoc., November 1913, Mr. J. E. Greiner, Consulting Engineer, has tabulated the live loads of 39 railroads, including all but one of the roads in Table XVI. Of the 39 roads thirteen are building bridges equal to E 60; four equal to E 57; seven equal to E 55; one equal to E 53; ten equal to E 50; two equal to E 47; one equal to E 45, and one equal to E 65.

Of the 39 roads considered 26 roads use the impact formula of the Am. Ry. Eng. Assoc.; and 24 roads use a tensile stress of 16,000 lb. per sq. in. The highest tensile stress is 18,000 lb.

TABLE XIV.

Data on Ties on Bridges.

Specifications.	Minimum Size and Spacing of Ties.			Data for Design.	
	Size.	Length.	Maximum Spacing.	Fiber Stress, Lb. per Sq. In.	Impact, Per Cent.
1. Am. Ry. Eng. Assoc.	....	10 ft.	6 in.	2,000	100
2. A. T. & St. F. R. R..	8 in. × 8 in.	12 ft.	12 in. centers	1,400	none
3. B. & O. R. R.	8 in. × 8 in.	9 ft.	6 in.	1,000	none
4. B. & M. R. R.	....	10 ft.	6 in.	2,000	100
5. C. M. & St. P. Ry...	....	10 ft.	6 in.	2,000	100
6. C. R. I. & P. R. R...	....	....	....	....	....
7. Common Standard...	8 in. × 10 in.	....	4 in.	....	....
8. Cooper	....	....	....	1,000	none
9. Illinois Central R. R.	{6″ × 8″ flat Four lines of stringers)	10 ft.	....	1,500	none
10. K. C., M. & O. R. R.	8 in. × 10 in.	10 ft.	13 in. centers on edge	2,000	100
11. L. V. R. R.	....	....	6 in.	....	....
12. N. Y. Central Lines..	....	....	....	....	...
13. N. Y., N. H. & H. R. R.	....	10 ft.	6 in.	2,000	100
14. Penna. W. of Pittsburgh.	....	....	....	....	....
15. Nat. L. of Mexico...	....	....	4 in.	1,000	none
16. Can. Soc. C. E.	....	....	....	1,800	100

TABLE XV.

Data on Dead Loads.

Specifications.	Weight in Lb.				
	Timber.	Ballast.	Concrete.	Rails and Fastenings.	Total Weight of Floor, Lb.
2. A., T. & S. F. R. R.	4½	....	....	....	Timber Ballasted Deck 1,400
3. B. & O. R. R.	4½	....	130	100	....
4. B. & M. R. R.	4½	100	150	150	....
5. C. M. & St. P. Ry.	4½	100	150	150	....
7. Common Standard	....	....	....	....	500
8. Cooper	4½	110	....	....	400 min.
9. Illinois Central R. R....	4½ Creosoted 5	100	150	100	....
10. K. C., M. & O. R. R....	....	....	....	....	400
11. Lehigh Valley R. R.	4½	....	150	170	550
12. N. Y. Central R. R.	4½	120	150	150	600
13. N. Y., N. H. & H. R. R.	4½	100	150	150	....
14. Penna. W. of Pittsburgh	....	....	....	....	400
15. Nat. L. of Mexico	4	100	....	120	....
17. Dept. of R. R. of Canada	4	....	....	....	600

per sq. in. and the lowest is 15,000 lb. per sq. in. Of the 39 roads considered all except one use a concentrated system of engine loadings; one road, the Pennsylvania Lines West of Pittsburgh, uses a uniform load of 5,500 lb. per lineal foot of track and an excess load of 66,000 lb. on one axle; no road is using an equivalent uniform load. For data on the heaviest locomotives in service and the relative stresses due to these locomotives compared with E 50 loading see Table II.*

Mr. Greiner's conclusion is that E 50 bridges will safely carry all loads that can be carried without increasing the present vertical and horizontal clearances.

* Also see Table XVII.

TABLE XVI.

Live Loads for Railway Bridges.

Specification.	Engine. Type.	Engine. Weight in 1,000 Lb.	Special Loads. Weight per Track. Two Loads, Lb.	Special Loads. Spacing of Two Loads, Ft.	Impact.	Tensile Unit Stress in Lb.	Equivalent Loading in Terms of Tensile Stress.
2. A., T. & S. F. R. R.	Consol.	291.0	......	....	Cooper	....	E 60
3. B. & O. R. R. ...	E 50	225.0	60,000	....	A. R. E. A.	16,000	E 50
4. B. & M. R. R. ...	E 60	270.0	65,000	6	"	16,000	E 60
5. C. M. & St. P. Ry.	E 55[1] E 60[1]	247.5 270.0	68,750 75,000	7 7	" "	16,000	E 55[2] E 60[2]
6. C. R. I. & P. R. R.	E 55	247.5	68,750	7	"	16,000	E 55
7. Common Standard	E 55	247.5	.. ...	....	Launhardt	$8{,}500\left(1+\frac{\text{min.}}{\text{max.}}\right)$	E 55[3]
9. Illinois Central R. R.	E 55	247.5	......	....	$\frac{LL}{LL+DL}$	16,000	E 55[4]
10. K. C., M. & O. R. R.	E 45	202.5	56,250	7	A. R. E. A.	18,000	E 40
11. Lehigh Valley R. R.	E 60	270.0	75,000	$7\frac{1}{2}$	"	16,000	E 60
12. N. Y. Central ...	E 60	270.0	72,000	7	"	18,000	E 53
13. N. Y., N. H. & H. R. R.	E 60	270.0	65,000	6	"	16,000	E 60
14. Penna. W. of Pittsburgh	Excess[5]	.....	...	....	Launhardt	$7{,}000\left(1+\frac{\text{min.}}{\text{max.}}\right)$	E 65
15. Nat. L. of Mex. .	E 60	270.0	75,000	5	Cooper	....	E 55

1. C. M. & St. P. Ry. uses E 55 east of the Missouri River and E 60 west.
2. A uniform train load of 7,000 lb. per lin. ft. on ore roads.
3. A uniform train load of 5,000 lb. per lin. ft.
4. A uniform train load of 6,000 lb. per lin. ft.
5. Train load of 5,500 lb. per lin. ft. and excess load of 66,000 lb.

§9. **Impact.**—Ten of the sixteen specifications use the impact coefficient as given in section 9, $I = 300/(L + 300)$. Three specifications follow Cooper's method of using dead load unit stresses equal to twice the live load unit stresses, with different stresses for different members. Two specifications use Launhardt's formula, $P = S\left(1 + \frac{\text{min. stress}}{\text{max. stress}}\right)$ where P = allowable unit stress, and S = allowable unit stress for live load alone. One specification uses the impact formula, $I = \frac{\text{Live Load Stress}}{\text{Live Load Stress} + \text{Dead Load Stress}}$. In the paper referred to in section 7, Mr. Greiner found that 26 roads used the A. R. E. A. formula for impact.

§10 & 11. **Wind Loads.**—The wind loads given in the different specifications are variable and space will not permit going into detail. Most of the specifications require that the moving wind load on the loaded chord be considered as applied at 6 or 7 ft. above the top of the rail.

§13. **Centrifugal Force.**—Five of the sixteen specifications have the same requirement as in section 13. The centrifugal force of a body moving in a circular path is $C = W \cdot V^2/32 \cdot 2R$, where W = weight of live load per lineal foot; V = velocity of train in feet per second, and R = radius of curve in feet. For a speed of $60 - 2\frac{1}{2}D$, $C = 0.039W$ for a 1 degree curve; $C = 0.071W$ for a 2 degree curve; $C = 0.117W$ for a 4 degree curve, and $C = 0.143W$ for a 10 degree curve. Five specifications require that the centrifugal force be applied at 5 to $7\frac{1}{2}$ feet above the rail. Two specifications take the centrifugal force as $C = 0.03W \cdot D$, where W = equivalent weight of live load per lineal foot, and D = degree of curve; one takes $C = 0.02W \cdot D$, and two take $C = 0.045W \cdot D$. The K. C. M. & O. R. R. takes $C = W \cdot V^2/32 \cdot 2R$, where W = equivalent weight of live load per lineal foot, V = velocity of train in feet per second (calculated for 50 miles per hour), and R = radius of curve in feet. This gives $C = 0.029W \cdot D$.

§14. **Unit Stresses.**—For a comparison of the tensile unit stresses see Table XVI.

§22. **Alternate Stresses.**—Four of the sixteen specifications use the same specification as in section 22. Six specifications use Cooper's specification. "All members and their connections shall be designed to resist each kind of stress. Both of the stresses shall, however, be considered as increased by 0.8 of the least of the two stresses." One specification increases each stress by 0.60 of the lesser stress, one by 0.70, and two by 0.75. One specification uses Weyrauch's formula, $P = S\left(1 - \frac{\text{min. stress}}{2 \text{ max. stress}}\right)$, where P = allowable unit stress for alternate stresses, and S = allowable unit stress for live loads alone.

§26. **Net Sections.**—Section 26 is standard. In addition the method of calculating the net area of a riveted tension member is given in several specifications.

Cooper requires that "The rupture of a riveted tension member is to be considered as equally probable, either through a transverse line of rivet holes or through a zigzag line of rivet hôles, where the net section does not exceed by 30 per cent the net section along a transverse line."

The Baltimore & Ohio R. R. requires that "The greatest number of rivet holes that can be cut by a transverse plane, or come within one inch of the plane is to be deducted in calculating the net section."

The New York Central Lines require that "The net section of riveted members shall be the least area which can be obtained by deducting from the gross sectional area the areas of holes cut by any plane perpendicular to the axis of the member and parts of the areas of other holes on one side of the plane, within a distance of 4 inches, and which are on other gage lines than those of the holes cut by the plane, the parts being determined by the formula: $A(1 - p/4)$, in which A = the area of the hole, and p = the distance in inches of the center of the hole from the plane."

The Canadian Society of Civil Engineers requires "There shall be deducted from each member as many rivets as there are gage lines, unless the distance center to center of rivets measured in the diagonal direction is 40 per cent greater than their distance center to center of gage lines."

§29. **Plate Girders.**—Seven of the sixteen specifications require that plate girders be proportioned either by the moment of inertia of their net section; or by assuming that the flanges are concentrated at their centers of gravity; in which case one-eighth of the gross section of the web, if properly spliced, may be used as flange section. Six specifications require that the bending moment all be taken by the flanges. Two specifications require that the bending moment be taken by the flanges and that one-eighth of the gross section of the web be taken as flange area. One specification requires that plate girders with stiffeners be designed on the assumption that the flanges take all the bending moment, and that for plate girders without stiffeners one-eighth of the web may be considered as flange area.

§30. **Compression Flanges.**—Two specifications require that the flange angles shall contain at least one-half of the area of the flange. The specifications uniformly require that the compression flange shall have the same gross area as the tension flange.

§36. **Counters.**—Eight specifications require that counters be stiff members. Eight specifications permit adjustable counters and laterals.

§45. **Minimum Angles.**—Five specifications give $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$ as the minimum angle. Two specifications give $3'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$ as the minimum angle. One specification requires that the vertical leg be not less than $3\frac{1}{2}''$. One specification requires that connection angles for stringers and floorbeams be not less than $4'' \times 4'' \times \frac{5}{8}''$; one specification $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, and one specification $6'' \times 4'' \times \frac{5}{8}''$.

§59. **Expansion.**—Six specifications require that provision be made for an expansion of $\frac{1}{8}$ in. for each 10 ft. of span. Five specifications require that provision be made for a range in temperature of 150 degrees F.; one for 180 degrees F. Three specifications require that provision be made for an expansion of 1 in. in 100 ft.; one for an expansion of 1 in. in 70 ft.

§62. **Rollers.**—Six specifications require that rollers be at least 6 in. in diameter. Five specifications permit rollers 4 in. in diameter. One specification permits rollers 3 in. in diameter. Cooper requires that rollers for spans up to 100 ft. be $4\frac{7}{8}$ in., and that the diameter be increased 1 in. for each 10 ft. increase in span over 100 ft. The New York Central R. R. requires that rollers shall not have a less diameter in inches than 3 + 0.03 (span in feet).

§68. **Stringer Connection Angles.**—One specification requires that connection angles of stringers and floorbeams be not less than $4'' \times 4'' \times \frac{5}{8}''$; one specification $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, and one specification $6'' \times 4'' \times \frac{5}{8}''$.

§77. **Camber of Plate Girders.**—Four specifications require that plate girders more than 50 ft. long be cambered $\frac{1}{16}$ in. per 10 ft. of length. Two specifications require full camber. Two specifications require a camber of $\frac{1}{1500}$ the span. Two specifications require a camber of $\frac{1}{1000}$ the span. One specification requires a camber of $\frac{1}{8}$ in. per 10 ft. of length, one specification requires a camber of $\frac{1}{16}$ in. per 15 ft. of length. Four specifications do not require that plate girders be cambered.

§79. **Web Stiffeners.**—Seven specifications have the same specification as given in section 79. Two specifications require that stiffeners be spaced not to exceed depth of girder. The Baltimore & Ohio R. R. requires that stiffeners be spaced not to exceed depth of girder or 6 ft., and that for webs up to 6 ft. 6 in., stiffeners shall be $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles; for webs from 7 ft. to 7 ft. 6 in., stiffeners shall be $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles; for webs 8 ft. and over, stiffeners shall be $6'' \times 4'' \times \frac{3}{8}''$ angles. The New York Central Lines require that stiffeners be spaced not to exceed depth of girder or 5 ft. 6 in.; near ends of girders the spacing shall not exceed one-half the depth of girder or 3 ft. 6 in.

The New York Central Lines require that stiffeners shall have an outstanding leg not less than 2 inches plus $\frac{1}{30}$ the depth of the girder.

The Chicago, Milwaukee & St. Paul Ry. requires that stiffeners bearing against $6'' \times 6''$ flange angles shall be $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$; and against $8'' \times 8''$ flange angles shall be $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$.

§81. **Camber of Trusses.**—Six specifications require full camber as stated in section 81. Six specifications require that the upper chords be increased $\frac{1}{8}$ in. for each 10 ft. One specification requires that the upper chord be increased $\frac{1}{8}$ in. for each 15 ft. Two specifications require that trusses be cambered $\frac{1}{1500}$ the span. One specification requires that trusses be cambered $\frac{1}{1000}$ the span.

§82. **Rigid Members.**—All specifications require that hip verticals and the two end panels of bottom chords (two at each end) be stiff members. The Common Standard specifications (Harriman Lines) require that the bottom chords of bridges of less than 150 ft. span be stiff members. The Illinois Central R. R. requires that bridges with 6 panels or less shall have stiff lower chords. The New York Central Lines limit the specification for rigid members to spans less than 300 ft.

§83. **Eye-bars.**—Nine specifications permit bars to be out of line 1 in. in 16 ft. as in section 83. One specification permits bars to be out of line 1 in. in 8 ft.

Miscellaneous.—The following specifications are of interest.

Initial Stress.—Four of the sixteen specifications require that diagonals and struts be designed for an initial stress of 10,000 lb. in each diagonal.

Collision Strut.—Two of the sixteen specifications require collision struts.

Fastening Angles.—Two specifications require that angles must be fastened by both legs. Three specifications require that angles be fastened by both legs or only one leg will be considered effective. One specification requires that 75 per cent of the net area be considered effective where angles are fastened by one leg, and 90 per cent of the net area be considered effective where angles are fastened by both legs.

Calculating Dead Load Stresses.—One specification requires that all the dead load be considered as coming on the loaded chord. Two specifications require that three-fourths of the dead load be considered as coming on the loaded chord and one-fourth on the unloaded chord. Two specifications require that two-thirds of the dead load be considered as coming on the loaded chord and one-third on the unloaded chord. Two specifications require that the floor load shall be assumed as taken by the loaded chord, and the remainder of the dead load to be divided equally between the chords. The other specifications do not state where the dead load shall be applied.

Minimum Bar.—Three specifications require that the minimum bar shall have not less than 3 sq. in. cross section. One specification permits a minimum bar $1\frac{1}{2}$ in. square. One specification requires that an increase of 80 per cent in the live load shall not increase the stress in the counters more than 80 per cent. One specification has a similar clause with 70 per cent variation.

Paint.—The shop coat of paint as required by several specifications is as follows:

The New York Central Lines use red lead paint mixed by the following formula:—100 lb. pure red lead; 4 gallons pure open-kettle-boiled linseed oil; and not to exceed one-half pint of turpentine-japan drier.

The Boston & Maine R. R. and the New York, New Haven & Hartford R. R. use red lead paint made by mixing 32 lb. of red lead to one gallon of linseed oil.

The A. T. & S. F. Ry. gives steel work a shop coat of linseed oil; while the C. R. I. & P. R. R. uses linseed oil with 10 per cent of lamp black.

The Illinois Central R. R. uses red lead paint for a shop coat.

The Pennsylvania Lines West of Pittsburgh use a shop coat of pure linseed oil.

The Common Standard specifications require a shop coat of red lead.

GENERAL SPECIFICATIONS FOR STEEL RAILWAY BRIDGES.*

American Railway Engineering Association.

Fourth Edition, 1910.

STANDARD SPECIFICATIONS.

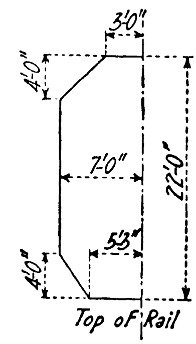
PART FIRST—DESIGN.

I. GENERAL.

1. **Materials.**—The material in the superstructure shall be structural steel, except rivets, and as may be otherwise specified.

2. **Clearances.**—When alinement is on tangent, clearances shall not be less than shown on the diagram; the height of rail shall, in all cases, be assumed as 6 in. The width shall be increased so as to provide the same minimum clearances on curves for a car 80 ft. long, 14 ft. high, and 60 ft. center to center of trucks, allowance being made for curvature and superelevation of rails.

3. **Spacing Trusses.**—The width center to center of girders and trusses shall in no case be less than one-twentieth of the effective span, nor less than is necessary to prevent overturning under the assumed lateral loading.



4. **Skew Bridges.**—Ends of deck plate girders and track stringers of skew bridges at abutments shall be square to the track, unless a ballasted floor is used.

5. **Floors.**—Wooden tie floors shall be secured to the stringers and shall be proportioned to carry the maximum wheel load, with 100 per cent impact, distributed over three ties, with fiber stress not to exceed 2,000 lb. per sq. in. Ties shall not be less than 10 ft. in length. They shall be spaced with not more than 6-in. openings; and shall be secured against bunching.

II. LOADS.

6. **Dead Load.**—The dead load shall consist of the estimated weight of the entire suspended structure. Timber shall be assumed to weigh $4\frac{1}{2}$ lb. per ft. B. M.; ballast 100 lb. per cu. ft., reinforced concrete 150 lb. per cu. ft., and rails and fastenings, 150 lb. per linear ft. of track.

†7. **Live Load.**—The live load, for each track, shall consist of two typical engines followed by a uniform load, according to Cooper's series, or a system of loading giving practically equivalent strains. The minimum loading to be Cooper's E-40, and the special loading, the diagram as shown in the following diagrams, that which gives the larger strains to be used.

†8. **Heavier Loading.**—Heavier loadings shall be proportional to the above diagrams on the same spacing.

9. **Impact.**—The dynamic increment of the live load shall be added to the maximum computed live load strains and shall be determined by the formula $I = S\dfrac{300}{L + 300}$,

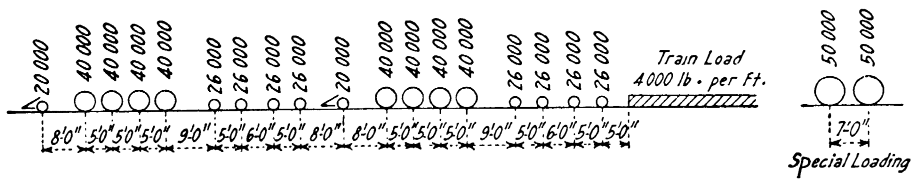
where I = impact or dynamic increment to be added to live-load strains.
S = computed maximum live-load strain.
L = loaded length of track in feet producing the maximum strain in the member. For bridges carrying more than one track, the aggregate length of all tracks producing the strain shall be used.

Impact shall not be added to strains produced by longitudinal, centrifugal and lateral or wind forces.

10. **Lateral Forces.**—All spans shall be designed for a lateral force on the loaded chord of 200 lb. per linear foot plus 10 per cent of the specified train load on one track, and 200 lb. per linear foot on the unloaded chord; these forces being considered as moving.

* Adopted by the American Railway Engineering Association, 1910.
† See Addendum, clause (a).

11. **Wind Force.**—Viaduct towers shall be designed for a force of 50 lb. per sq. ft. on one and one-half times the vertical projection of the structure unloaded; or 30 lb. per sq. ft. on the same surface plus 400 lb. per linear ft. of structure applied 7 ft. above the rail for assumed wind force on train when the structure is either fully loaded or loaded on either track with empty cars assumed to weigh 1,200 lb. per linear ft., whichever gives the larger strain.



12. **Longitudinal Force.**—Viaduct towers and similar structures shall be designed for a longitudinal force of 20 per cent of the live load applied at the top of the rail.

13. Structures located on curves shall be designed for the centrifugal force of the live load applied at the top of the high rail. The centrifugal force shall be considered as live load and be derived from the speed in miles per hour given by the expression $60 - 2\frac{1}{2}D$, where "D" = degree of curve.

III. UNIT STRESSES AND PROPORTION OF PARTS.

14. **Unit Stresses.**—All parts of structures shall be so proportioned that the sum of the maximum stresses produced by the foregoing loads shall not exceed the following amounts in pounds per sq. in., except as modified in paragraphs 22 to 25:

15. **Tension.**—Axial tension on net section 16,000

16. **Compression.**—Axial compression on gross section of columns . . . . $16,000 - 70\frac{l}{r}$

with a maximum of 14,000
where "l" is the length of the member in inches, and "r" is the least radius of gyration in inches.
Direct compression on steel castings 16,000

17. **Bending.**—Bending: on extreme fibers of rolled shapes, built sections, girders and steel castings; net section 16,000
on extreme fibers of pins 24,000

18. **Shearing.**—Shearing: shop driven rivets and pins 12,000
field driven rivets and turned bolts 10,000
plate girder webs; gross section 10,000

19. **Bearing.**—Bearing: shop driven rivets and pins 24,000
field driven rivets and turned bolts 20,000
expansion rollers; per linear inch . . . . $600d$
where "d" is the diameter of the roller in inches.
on masonry 600

20. **Limiting Length of Members.**—The lengths of main compression members shall not exceed 100 times their least radius of gyration, and those for wind and sway bracing 120 times their least radius of gyration.

21. The lengths of riveted tension members in horizontal or inclined positions shall not exceed 200 times their radius of gyration about the horizontal axis. The horizontal projection of the unsupported portion of the member is to be considered as the effective length.

22. **Alternate Stresses.**—Members subject to alternate stresses of tension and compression shall be proportioned for the stresses giving the largest section. If the alternate stresses occur in succession during the passage of one train, as in stiff counters, each stress shall be increased by 50 per cent of the smaller. The connections shall in all cases be proportioned for the sum of the stresses.

23. Wherever the live and dead load stresses are of opposite character, only two-thirds of the dead load stresses shall be considered as effective in counteracting the live load stress.

24. **Combined Stresses.**—Members subject to both axial and bending stresses shall be proportioned so that the combined fiber stresses will not exceed the allowed axial stress.

25. For stresses produced by longitudinal and lateral or wind forces combined with those from live and dead loads and centrifugal force, the unit stress may be increased 25 per cent over

those given above; but the section shall not be less than required for live and dead loads and centrifugal force.

26. **Net Section at Rivets.**—In proportioning tension members the diameter of the rivet holes shall be taken $\frac{1}{8}$-in. larger than the nominal diameter of the rivet.

27. **Rivets.**—In proportioning rivets the nominal diameter of the rivet shall be used.

28. **Net Section at Pins.**—Pin-connected riveted tension members shall have a net section through the pin-hole at least 25 per cent in excess of the net section of the body of the member, and the net section back of the pin-hole, parallel with the axis of the member, shall be not less than the net section of the body of the member.

29. **Plate Girders.**—Plate girders shall be proportioned either by the moment of inertia of their net section; or by assuming that the flanges are concentrated at their centers of gravity; in which case one-eighth of the gross section of the web, if properly spliced, may be used as flange section. The thickness of web plates shall be not less than $\frac{1}{160}$ of the unsupported distance between flange angles (see 38).

30. **Compression Flange.**—The gross section of the compression flanges of plate girders shall not be less than the gross section of the tension flanges; nor shall the stress per sq. in. in the compression flange of any beam or girder exceed $16{,}000 - 200\frac{l}{b}$, when flange consists of angles only or if cover consists of flat plates, or $16{,}000 - 150\frac{l}{b}$, if cover consists of a channel section, where l = unsupported distance and b = width of flange.

31. **Flange Rivets.**—The flanges of plate girders shall be connected to the web with a sufficient number of rivets to transfer the total shear at any point in a distance equal to the effective depth of the girder at that point combined with any load that is applied directly on the flange. The wheel loads, where the ties rest on the flanges, shall be assumed to be distributed over three ties.

32. **Depth Ratios.**—Trusses shall preferably have a depth of not less than one-tenth of the span. Plate girders and rolled beams, used as girders, shall preferably have a depth of not less than one-twelfth of the span. If shallower trusses, girders or beams are used, the section shall be increased so that the maximum deflection will not be greater than if the above limiting ratios had not been exceeded.

IV. DETAILS OF DESIGN.

GENERAL REQUIREMENTS.

33. **Open Sections.**—Structures shall be so designed that all parts will be accessible for inspection, cleaning and painting.

34. **Pockets.**—Pockets or depressions which would hold water shall have drain holes, or be filled with waterproof material.

35. **Symmetrical Sections.**—Main members shall be so designed that the neutral axis will be as nearly as practicable in the center of section, and the neutral axes of intersecting main members of trusses shall meet at a common point.

36. **Counters.**—Rigid counters are preferred; and where subject to reversal of stress shall preferably have riveted connections to the chords. Adjustable counters shall have open turnbuckles.

37. **Strength of Connections.**—The strength of connections shall be sufficient to develop the full strength of the member, even though the computed stress is less, the kind of stress to which the member is subjected being considered.

38. **Minimum Thickness.**—The minimum thickness of metal shall be $\frac{3}{8}$-in., except for fillers.

39. **Pitch of Rivets.**—The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3 in. for $\frac{7}{8}$-in. rivets and $2\frac{5}{8}$ in. for $\frac{3}{4}$-in. rivets. The maximum pitch in the line of stress for members composed of plates and shapes shall be 6 in. for $\frac{7}{8}$-in. rivets and 5 in. for $\frac{3}{4}$-in. rivets. For angles with two gage lines and rivets staggered the maximum shall be twice the above in each line. Where two or more plates are used in contact, rivets not more than 12 in. apart in either direction shall be used to hold the plates well together. In tension members, composed of two angles in contact, a pitch of 12 in. will be allowed for riveting the angles together.

40. **Edge Distance.**—The minimum distance from the center of any rivet hole to a sheared edge shall be $1\frac{1}{2}$ in. for $\frac{7}{8}$-in. rivets and $1\frac{1}{4}$ in. for $\frac{3}{4}$-in. rivets, and to a rolled edge $1\frac{1}{4}$ in. and $1\frac{1}{8}$ in., respectively. The maximum distance from any edge shall be eight times the thickness of the plate, but shall not exceed 6 in.

41. **Maximum Diameter.**—The diameter of the rivets in any angle carrying calculated stress shall not exceed one-quarter the width of the leg in which they are driven. In minor parts ⅞-in. rivets may be used in 3-in. angles, and ¾-in. rivets in 2½-in. angles.

42. **Long Rivets.**—Rivets carrying calculated stress and whose grip exceeds four diameters shall be increased in number at least one per cent for each additional $\frac{1}{16}$-in. of grip.

43. **Pitch at Ends.**—The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets, for a length equal to one and one-half times the maximum width of member.

44. **Compression Members.**—In compression members the metal shall be concentrated as much as possible in webs and flanges. The thickness of each web shall be not less than one-thirtieth of the distance between its connections to the flanges. Cover plates shall have a thickness not less than one-fortieth of the distance between rivet lines.

45. **Minimum Angles.**—Flanges of girders and built members without cover plates shall have a minimum thickness of one-twelfth of the width of the outstanding leg.

46. **Tie-Plates.**—The open sides of compression members shall be provided with lattice and shall have tie-plates as near each end as practicable. Tie-plates shall be provided at intermediate points where the lattice is interrupted. In main members the end tie-plates shall have a length not less than the distance between the lines of rivets connecting them to the flanges, and intermediate ones not less than one-half this distance. Their thickness shall not be less than one-fiftieth of the same distance.

47. **Lattice.**—The latticing of compression members shall be proportioned to resist the shearing stresses corresponding to the allowance for flexure for uniform load provided in the column formula in paragraph 16 by the term $70\frac{l}{r}$. The minimum width of lattice bars shall be 2½ in. for ⅞-in. rivets, 2¼ in. for ¾-in. rivets, and 2 in. if ⅝-in. rivets are used. The thickness shall not be less than one-fortieth of the distance between end rivets for single lattice, and one-sixtieth for double lattice. Shapes of equivalent strength may be used.

48. Three-fourths-inch rivets shall be used for latticing flanges less than 2½ in. wide, and ¾-in. for flanges from 2½ to 3½ in. wide; ⅞-in. rivets shall be used in flanges 3½ in. and over, and lattice bars with at least two rivets shall be used for flanges over 5 in. wide.

49. The inclination of lattice bars with the axis of the member shall be not less than 45 degrees, and when the distance between rivet lines in the flanges is more than 15 in., if single rivet bar is used, the lattice shall be double and riveted at the intersection.

50. Lattice bars shall be so spaced that the portion of the flange included between their connections shall be as strong as the member as a whole.

51. **Faced Joints.**—Abutting joints in compression members when faced for bearing shall be spliced on four sides sufficiently to hold the connecting members accurately in place. All other joints in riveted work, whether in tension or compression, shall be fully spliced.

52. **Pin Plates.**—Pin-holes shall be reinforced by plates where necessary, and at least one plate shall be as wide as the flanges will allow and be on the same side as the angles. They shall contain sufficient rivets to distribute their portion of the pin pressure to the full cross-section of the member.

53. **Forked Ends.**—Forked ends on compression members will be permitted only where unavoidable; where used, a sufficient number of pin plates shall be provided to make the jaws of twice the sectional area of the member. At least one of these plates shall extend to the far edge of the farthest tie-plate, and the balance to the far edge of the nearest tie-plate, but not less than 6 in. beyond the near edge of the farthest plate.

54. **Pins.**—Pins shall be long enough to insure a full bearing of all the parts connected upon the turned body of the pin. They shall be secured by chambered nuts or be provided with washers if solid nuts are used. The screw ends shall be long enough to admit of burring the threads.

55. Members packed on pins shall be held against lateral movement.

56. **Bolts.**—Where members are connected by bolts, the turned body of these bolts shall be long enough to extend through the metal. A washer at least ¼-in. thick shall be used under the nut. Bolts shall not be used in place of rivets except by special permission. Heads and nuts shall be hexagonal.

57. **Indirect Splices.**—Where splice plates are not in direct contact with the parts which they connect, rivets shall be used on each side of the joint in excess of the number theoretically required to the extent of one-third of the number for each intervening plate.

58. **Fillers.**—Rivets carrying stress and passing through fillers shall be increased 50 per cent in number; and the excess rivets, when possible, shall be outside of the connected member.

59. **Expansion.**—Provision for expansion to the extent of ⅛-in. for each 10 ft. shall be made for all bridge structures. Efficient means shall be provided to prevent excessive motion at any one point.

60. **Expansion Bearings.**—Spans of 80 ft. and over resting on masonry shall have turned rollers or rockers at one end; and those of less length shall be arranged to slide on smooth surfaces. These expansion bearings shall be designed to permit motion in one direction only.

61. **Fixed Bearings.**—Fixed bearings shall be firmly anchored to the masonry.

62. **Rollers.**—Expansion rollers shall be not less than 6 in. in diameter. They shall be coupled together with substantial side bars, which shall be so arranged that the rollers can be readily cleaned. Segmental rollers shall be geared to the upper and lower plates.

63. **Bolsters.**—Bolsters or shoes shall be so constructed that the load will be distributed over the entire bearing. Spans of 80 ft. or over shall have hinged bolsters at each end.

64. **Wall Plates.**—Wall plates may be cast or built up; and shall be so designed as to distribute the load uniformly over the entire bearing. They shall be secured against displacement.

65. **Anchorage.**—Anchor bolts for viaduct towers and similar structures shall be long enough to engage a mass of masonry the weight of which is at least one and one-half times the uplift.

66. **Inclined Bearings.**—Bridges on an inclined grade without pin shoes shall have the sole plates beveled so that the masonry and expansion surfaces may be level.

FLOOR SYSTEMS.

67. **Floorbeams.**—Floorbeams shall preferably be square to the trusses or girders. They shall be riveted directly to the girders or trusses or may be placed on top of deck bridges.

68. **Stringers.**—Stringers shall preferably be riveted to the webs of all intermediate floorbeams by means of connection angles not less than $\frac{1}{2}$-in. in thickness. Shelf angles or other supports provided to support the stringer during erection shall not be considered as carrying any of the reaction.

69. **Stringer Frames.**—Where end floorbeams cannot be used, stringers resting on masonry shall have cross frames near their ends. These frames shall be riveted to girders or truss shoes where practicable.

BRACING.

70. **Rigid Bracing.**—Lateral, longitudinal and transverse bracing in all structures shall be composed of rigid members.

71. **Portals.**—Through truss spans shall have riveted portal braces rigidly connected to the end posts and top chords. They shall be as deep as the clearance will allow.

72. **Transverse Bracing.**—Intermediate transverse frames shall be used at each panel of through spans having vertical truss members where the clearance will permit.

73. **End Bracing.**—Deck spans shall have transverse bracing at each end proportioned to carry the lateral load to the support.

74. **Laterals.**—The minimum sized angle to be used in lateral bracing shall be $3\frac{1}{2}$ by 3 by $\frac{3}{8}$-in. Not less than three rivets through the end of the angles shall be used at the connection.

75. Lateral bracing shall be far enough below the flange to clear the ties.

76. **Tower Struts.**—The struts at the foot of viaduct towers shall be strong enough to slide the movable shoes when the track is unloaded.

PLATE GIRDERS.

77. **Camber.**—If desired, plate girder spans over 50 ft. in length shall be built with camber at a rate of $\frac{1}{16}$-in. per 10 ft. of length.

78. **Top Flange Cover.**—Where flange plates are used, one cover plate of top flange shall extend the whole length of the girder.

79. **Web Stiffeners.**—There shall be web stiffeners, generally in pairs, over bearings, at points of concentrated loading, and at other points where the thickness of the web is less than $\frac{1}{60}$ of the unsupported distance between flange angles. The distance between stiffeners shall not exceed that given by the following formula, with a maximum limit of six feet (and not greater than the clear depth of the web):

$$d = \frac{t}{40}(12{,}000 - s),$$

Where d = clear distance, between stiffeners of flange angles.
t = thickness of web.
s = shear per sq. in.

The stiffeners at ends and at points of concentrated loads shall be proportioned by the formula of paragraph 16, the effective length being assumed as one-half the depth of girders. End stiffeners and those under concentrated loads shall be on fillers and have their outstanding legs as wide as the flange angles will allow and shall fit tightly against them. Intermediate stiffeners may be

offset or on fillers, and their outstanding legs shall be not less than one-thirtieth of the depth of girder plus 2 in.

80. **Stays for Top Flanges.**—Through plate girders shall have their top flanges stayed at each end of every floorbeam, or in case of solid floors, at distances not exceeding 12 ft., by knee braces or gusset plates.

TRUSSES.

81. **Camber.**—Truss spans shall be given a camber by so proportioning the length of the members that the stringers will be straight when the bridge is fully loaded.

82. **Rigid Members.**—Hip verticals and similar members, and the two end panels of the bottom chords of single track pin-connected trusses shall be rigid.

83. **Eye-bars.**—The eye-bars composing a member shall be so arranged that adjacent bars shall not have their surfaces in contact; they shall be as nearly parallel to the axis of the truss as possible, the maximum inclination of any bar being limited to one inch in 16 ft.

84. **Pony Trusses.**—Pony trusses shall be riveted structures, with double webbed chords, and shall have all web members latticed or otherwise effectively stiffened.

PART SECOND—MATERIALS AND WORKMANSHIP.

V. MATERIAL.

85. **Steel.**—Steel shall be made by the open-hearth process.

86. **Properties.**—The chemical and physical properties shall conform to the following limits:

Elements Considered.	Structural Steel.	Rivet Steel.	Steel Castings.
Phosphorus, max.. { Basic...	0.04 per cent	0.04 per cent	0.05 per cent
Phosphorus, max.. { Acid....	0.06 per cent	0.04 per cent	0.08 per cent
Sulphur, maximum..........	0.05 per cent	0.04 per cent	0.05 per cent
Ultimate tensile strength. Pounds per square inch......	Desired. 60,000	Desired. 50,000	Not less than 65,000
Elong., min. %, in 8″, Fig. 1	1,500,000* / Ult. tensile strength	1,500,000 / Ult. tensile strength	15 per cent
Elong., min. %, in 2″, Fig. 2..	22		
Character of Fracture.......	Silky	Silky	Silky or fine granular
Cold Bends without Fracture.	180° flat†	180° flat‡	90° $d = 3t$

The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

87. In order that the ultimate strength of full-sized annealed eye-bars may meet the requirements of paragraph 163, the ultimate strength in test specimens may be determined by the manufacturers; all other tests than those for ultimate strength shall conform to the above requirements.

88. **Allowable Variations.**—If the ultimate strength varies more than 4,000 lb. from that desired, a retest shall be made on the same gage, which, to be acceptable, shall be within 5,000 lb. of the desired ultimate.

89. **Chemical Analyses.**—Chemical determinations of the percentages of carbon, phosphorus, sulphur and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for by the purchaser, in which case an excess of 25 per cent above the required limits will be permitted.

90. **Specimens.**—Plate, shape and bar specimens for tensile and bending tests shall be made by cutting coupons from the finished product, which shall have both faces rolled and both edges milled to the form shown by Fig. 1; or with both edges parallel; or they may be turned to a diameter of ¾-in. for a length of at least 9 in., with enlarged ends.

91. Rivet rods shall be tested as rolled.

* See paragraph 96. † See paragraphs 97, 98, and 99. ‡ See paragraph 100.

92. Pin and roller specimens shall be cut from the finished rolled or forged bar, in such manner that the center of the specimen shall be one inch from the surface of the bar. The specimen for tensile test shall be turned to the form shown by Fig. 2. The specimen for bending test shall be one inch by ½-in. in section.

93. For steel castings the number of tests will depend on the character and importance of the castings. Specimens shall be cut cold from coupons molded and cast on some portion of one or more castings from each melt or from the sink heads, if the heads are of sufficient size. The coupon or sink head, so used, shall be annealed with the casting before it is cut off. Test specimens to be of the form prescribed for pins and rollers.

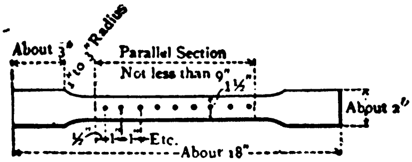


FIG. 1

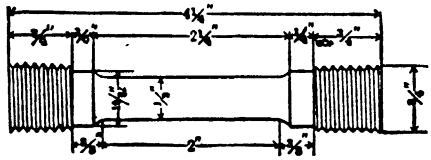


FIG. 2.

94. **Specimens of Rolled Steel.**—Rolled steel shall be tested in the condition in which it comes from the rolls.

95. **Number of Tests.**—At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing ⅜-in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

96. **Modification in Elongation.**—A deduction of 1 per cent will be allowed from the specified percentage for elongation, for each ⅛-in. in thickness above ¾-in.

97. **Bending Tests.**—Bending tests may be made by pressure or by blows. Plates, shapes and bars less than one inch thick shall bend as called for in paragraph 86.

98. **Thick Material.**—Full-sized material for eye-bars and other steel one inch thick and over, tested as rolled, shall bend cold 180 degrees around a pin, the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of bend.

99. **Bending Angles.**—Angles ¾-in. and less in thickness shall open flat, and angles ½-in. and less in thickness shall bend shut, cold, under blows of a hammer, without sign of fracture. This test shall be made only when required by the inspector.

100. **Nicked Bends.**—Rivet steel, when nicked and bent around a bar of the same diameter as the rivet rod, shall give a gradual break and a fine silky uniform fracture.

101. **Finish.**—Finished material shall be free from injurious seams, flaws, cracks, defective edges or other defects, and have a smooth, uniform and workmanlike finish. Plates 36 in. in width and under shall have rolled edges.

102. **Melt Numbers.**—Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it. Steel for pins and rollers shall be stamped on the end. Rivet and lattice steel and other small parts may be bundled with the above marks on an attached metal tag.

103. **Defective Material.**—Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, will be rejected at the shop and shall be replaced by the manufacturer at his own cost.

104. **Variation in Weight.**—A variation in cross-section or weight of each piece of steel of more than 2½ per cent from that specified will be sufficient cause for rejection, except in case of sheared plates, which will be covered by the following permissible variations, which are to apply to single plates, when ordered to weight:

105. Plates 12½ lb. per sq. ft. or heavier:

(a) Up to 100 in. wide, 2½ per cent above or below the prescribed weight.

(b) One hundred inches wide and over, 5 per cent above or below.

106. Plates under 12½ lb. per sq. ft.:

(a) Up to 75 in. wide, 2½ per cent above or below.

(b) Seventy-five inches and up to 100 in. wide, 5 per cent above or 3 per cent below.

(c) One hundred inches wide and over, 10 per cent above or 3 per cent below.

107. Plates when ordered to gage will be accepted if they measure not more than 0.01 in. below the ordered thickness.

108. An excess over the nominal weight, corresponding to the dimensions on the order, will be allowed for each plate, if not more than that shown in the following table, one cu. in. of rolled steel being assumed to weigh 0.2833 lb.:

Thickness Ordered.	Nominal Weights.	Width of Plate.			
		Up to 75″.	75″ and up to 100″.	100″ and up to 115″.	Over 115″.
$\frac{1}{4}$-inch	10.20 lb.	10 per cent	14 per cent	18 per cent	....
$\frac{5}{16}$ "	12.75 "	8 "	12 "	16 "	....
$\frac{3}{8}$ "	15.30 "	7 "	10 "	13 "	17 per cent
$\frac{7}{16}$ "	17.85 "	6 "	8 "	10 "	13 "
$\frac{1}{2}$ "	20.40 "	5 "	7 "	9 "	12 "
$\frac{9}{16}$ "	22.95 "	4½ "	6½ "	8½ "	11 "
$\frac{5}{8}$ "	25.50 "	4 "	6 "	8 "	10 "
Over $\frac{5}{8}$ "	....	3½ "	5 "	6½ "	9 "

109. **Cast-Iron.**—Except where chilled iron is specified, castings shall be made of tough gray iron, with sulphur not over 0.10 per cent. They shall be true to pattern, out of wind and free from flaws and excessive shrinkage. If tests are demanded, they shall be made on the "Arbitration Bar" of the American Society for Testing Materials, which is a round bar 1¼ in. in diameter and 15 in. long. The transverse test shall be made on a supported length of 12 in. with load at middle. The minimum breaking load so applied shall be 2,900 lb., with a deflection of at least $\frac{1}{10}$ in. before rupture.

110. **Wrought-Iron.**—Wrought-iron shall be double-rolled, tough, fibrous and uniform in character. It shall be thoroughly welded in rolling and be free from surface defects. When tested in specimens of the form of Fig. 1, or in full-sized pieces of the same length, it shall show an ultimate strength of at least 50,000 lb. per sq. in., an elongation of at least 18 per cent in 8 in., with fracture wholly fibrous. Specimens shall bend cold, with the fiber, through 135 degrees, without sign of fracture, around a pin the diameter of which is not over twice the thickness of the piece tested. When nicked and bent, the fracture shall show at least 90 per cent fibrous.

VI. INSPECTION AND TESTING AT THE MILLS.

111. **Mill Orders.**—The purchaser shall be furnished complete copies of mill orders, and no material shall be rolled nor work done before the purchaser has been notified where the orders have been placed, so that he may arrange for the inspection.

112. **Facilities for Inspection.**—The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of all material at the mill where it is manufactured. He shall furnish a suitable testing machine for testing the specimens as well as prepare the pieces for the machine, free of cost.

113. **Access to Mills.**—When an inspector is furnished by the purchaser to inspect material at the mills, he shall have full access, at all times, to all parts of mills where material to be inspected by him is being manufactured.

VII. WORKMANSHIP.

114. **General.**—All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works. Material arriving from the mills shall be protected from the weather and shall have clean surfaces before being worked in the shops.

115. **Straightening.**—Material shall be thoroughly straightened in the shop, by methods that will not injure it, before being laid off or worked in any way.

116. **Finish.**—Shearing and chipping shall be neatly and accurately done and all portions of the work exposed to view neatly finished.

117. **Size of Rivets.**—The size of rivets, called for on the plans, shall be understood to mean the actual size of the cold rivet before heating.

118. **Rivet Holes.**—When general reaming is not required, the diameter of the punch shall not be more than $\frac{1}{16}$-in. greater than the diameter of the rivet; nor the diameter of the die more than $\frac{1}{8}$-in. greater than the diameter of the punch. Material more than $\frac{3}{4}$-in. thick shall be sub-punched and reamed or drilled from the solid.

119. **Punching.**—Punching shall be accurately done. Drifting to enlarge unfair holes will not be allowed. If the holes must be enlarged to admit the rivet, they shall be reamed. Poor matching of holes will be cause for rejection.

120. **Reaming.**—Where sub-punching and reaming are required, the punch used shall have a diameter not less than $\frac{3}{16}$-in. smaller than the nominal diameter of the rivet. Holes shall then be reamed to a diameter not more than $\frac{1}{16}$-in. larger than the nominal diameter of the rivet. (See 135.)

121. **Reaming after Assembling.***—[When general reaming is required it shall be done after the pieces forming one built member are assembled and so firmly bolted together that the surfaces shall be in close contact. If necessary to take the pieces apart for shipping and handling, the respective pieces reamed together shall be so marked that they may be reassembled in the same position in the final setting up. No interchange of reamed parts will be permitted.]

122. Reaming shall be done with twist drills and without using any lubricant.

123. The outside burrs on reamed holes shall be removed to the extent of making a $\frac{1}{16}$-in. fillet.

124. **Assembling.**—Riveted members shall have all parts well pinned up and firmly drawn together with bolts, before riveting is commenced. Contact surfaces to be painted. (See 152.)

125. **Lattice Bars.**—Lattice bars shall have neatly rounded ends, unless otherwise called for.

126. **Web Stiffeners.**—Stiffeners shall fit neatly between flanges of girders. Where tight fits are called for, the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

127. **Splice Plate and Fillers.**—Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$-in. of flange angles.

128. **Web Plates.**—Web plates of girders, which have no cover plates, shall be flush with the backs of angles or project above the same not more than $\frac{1}{8}$-in., unless otherwise called for. When web plates are spliced, not more than $\frac{1}{4}$-in. clearance between ends of plates will be allowed.

129. **Floorbeams and Stringers.**—The main sections of floorbeams and stringers shall be milled to exact length after riveting and the connection angles accurately set flush and true to the milled ends †[or if required by the purchaser the milling shall be done after the connection angles are riveted in place, milling to extend over the entire face of the member]. The removal of more than $\frac{3}{32}$-in. from the thickness of the connection angles will be cause for rejection.

130. **Riveting.**—Rivets shall be uniformly heated to a light cherry red heat in a gas or oil furnace so constructed that it can be adjusted to the proper temperature. They shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

131. Rivets shall look neat and finished, with heads of approved shape, full and of equal size. They shall be central on shank and grip the assembled pieces firmly. Recupping and calking will not be allowed. Loose, burned or otherwise defective rivets shall be cut out and replaced. In cutting out rivets, great care shall be taken not to injure the adjacent metal. If necessary, they shall be drilled out.

132. **Turned Bolts.**—Wherever bolts are used in place of rivets which transmit shear, the holes shall be reamed parallel and the bolts shall make a driving fit with the threads entirely outside of the holes. A washer not less than $\frac{1}{4}$-in. thick shall be used under nut.

133. **Members to be Straight.**—The several pieces forming one built member shall be straight and fit closely together, and finished members shall be free from twists, bends or open joints.

134. **Finish of Joints.**—Abutting joints shall be cut or dressed true and straight and fitted close together, especially where open to view. In compression joints, depending on contact bearing, the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

135. **Field Connections.**—Holes for floorbeam and stringer connections shall be sub-punched and reamed according to paragraph 120, to a steel templet not less than one inch thick. ‡[If required, all other field connections, except those for laterals and sway bracing, shall be assembled in the shop and the unfair holes reamed; and when so reamed the pieces shall be match-marked before being taken apart.]

136. **Eye-Bars.**—Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of heads will be determined by the dies in use

* See Addendum, clause (d).
† See Addendum, clause (f).
‡ See Addendum, clause (e).

at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body when tested to rupture. The thickness of head and neck shall not vary more than $\frac{1}{16}$-in. from that specified. (See 163.)

137. **Boring Eye-Bars.**—Before boring, each eye-bar shall be properly annealed and carefully straightened. Pin-holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that, when placed together, pins $\frac{1}{32}$-in. smaller in diameter than the pin-holes can be passed through the holes at both ends of the bars at the same time without forcing.

138. **Pin-Holes.**—Pin-holes shall be bored true to gages, smooth and straight; at right angles to the axis of the member and parallel to each other, unless otherwise called for. The boring shall be done after the member is riveted up.

139. The distance center to center of pin-holes shall be correct within $\frac{1}{32}$-in., and the diameter of the holes not more than $\frac{1}{50}$-in. larger than that of the pin, for pins up to 5-in. diameter, and $\frac{1}{32}$-in. for larger pins.

140. **Pins and Rollers.**—Pins and rollers shall be accurately turned to gages and shall be straight and smooth and entirely free from flaws.

141. **Screw Threads.**—Screw threads shall make tight fits in the nuts and shall be U. S. standard, except above the diameter of 1⅜ in., when they shall be made with six threads per inch.

142. **Annealing.**—Steel, except in minor details, which has been partially heated, shall be properly annealed.

143. **Steel Castings.**—Steel castings shall be free from large or injurious blowholes and shall be annealed.

144. **Welds.**—Welds in steel will not be allowed.

145. **Bed Plates.**—Expansion bed plates shall be planed true and smooth. Cast wall plates shall be planed top and bottom. The finishing cut of the planing tool shall be fine and correspond with the direction of expansion.

146. **Pilot Nuts.**—Pilot and driving nuts shall be furnished for each size of pin, in such numbers as may be ordered.

147. **Field Rivets.**—Field rivets shall be furnished to the amount of 15 per cent plus ten rivets in excess of the nominal number required for each size.

148. **Shipping Details.**—Pins, nuts, bolts, rivets and other small details shall be boxed or crated.

149. **Weight.**—The scale weight of every piece and box shall be marked on it in plain figures.

150. **Finished Weight.**—Payment for pound price contracts shall be by scale weight. No allowance over 2 per cent of the total weight of the structure as computed from the plans will be allowed for excess weight.

VIII. SHOP PAINTING.

*151. **Cleaning.**—Steel work, before leaving the shop, shall be thoroughly cleaned and given one good coating of pure linseed oil, or such paint as may be called for, well worked into all joints and open spaces.

152. **Contact Surfaces.**—In riveted work, the surfaces coming in contact shall each be painted before being riveted together.

153. **Inaccessible Surfaces.**—Pieces and parts which are not accessible for painting after erection, including tops of stringers, eye-bar heads, ends of posts and chords, etc., shall have an additional coat of paint before leaving the shop.

154. **Condition of Surfaces.**—Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

155. **Machine-Finished Surfaces.**—Machine-finished surfaces shall be coated with white lead and tallow before shipment or before being put out into the open air.

IX. INSPECTION AND TESTING AT THE SHOPS.

156. **Facilities for Inspection.**—The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of workmanship at the shop where material is manufactured. He shall furnish a suitable testing machine for testing full-sized members, if required.

157. **Starting Work.**—The purchaser shall be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect material and workmanship.

158. **Access to Shop.**—When an inspector is furnished by the purchaser, he shall have full access, at all times, to all parts of the shop where material under his inspection is being manufactured.

159. **Accepting Material.**—The inspector shall stamp each piece accepted with a private mark. Any piece not so marked may be rejected at any time and at any stage of the work. If the in-

* See Addendum, clause (b).

spector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the purchaser.

160. **Shop Plans.**—The purchaser shall be furnished complete shop plans.

161. **Shipping Invoices.**—Complete copies of shipping invoices shall be furnished to the purchaser with each shipment. These shall show the scale weights of individual pieces.

X. FULL-SIZED TESTS.

162. **Eye-Bar Tests.**—Full-sized tests on eye-bars and similar members, to prove the workmanship, shall be made at the manufacturer's expense, and shall be paid for by the purchaser at contract price, if the tests are satisfactory. If the tests are not satisfactory, the members represented by them will be rejected.

163. In eye-bar tests, the minimum ultimate strength shall be 55,000 lb. per sq. in. The elongation in 10 ft., including fracture, shall be not less than 15 per cent. Bars shall generally break in the body and the fracture shall be silky or fine granular, and the elastic limit as indicated by the drop of the mercury shall be recorded. Should a bar break in the head and develop the specified elongation, ultimate strength and character of fracture, it shall not be cause for rejection, provided not more than one-third of the total number of bars break in the head (see 136).

ADDENDUM TO GENERAL SPECIFICATIONS FOR STEEL RAILWAY BRIDGES.

POINTS TO BE SPECIFICALLY DETERMINED BY BUYERS WHEN SOLICITING PROPOSALS FOR STEEL RAILWAY BRIDGES.

When general detail drawings are not furnished for the use of bidders specific answers should be given to questions a, b and c, below.

Specific answers should also be given to questions d, e and f if the class of work described in any of the paragraphs there referred to is desired. If these features are not specifically demanded, the unbracketed paragraphs will be construed to define the kind of work desired.

(a) What class of live load shall be used? (Pars. 7 and 8.)
(b) Shall linseed oil or paint be used? If paint, what kind? (Par. 151.)
(c) Shall contractor furnish floor bolts?
(d) Shall general reaming be done? (Par. 121.)
(e) Shall field connections be assembled at the shop? (Par. 135.)
(f) Shall floor connection angles be milled after riveting? (Par. 129.)

INSTRUCTIONS FOR THE DESIGN OF RAILWAY BRIDGES.*

The following instructions for the design of the details of railway bridges have been prepared by the engineering department of the Chicago, Milwaukee & St. Paul Railway, 1912.

RIVETS AND RIVET SPACING.—1. For conventional signs, actual sizes of heads and lengths of field rivets for various grips, see Fig. 10, Chap. XII, and Table 109, Part II.

2. **Size.**—Rivets for steel bridge work shall usually be ⅞ in. diameter, except where limited by size of material. In very heavy work, where rivets of long grip are required, such as in the drums of draw spans, 1 in. rivets are preferable.

3. **Flattened.**—Rivet heads are not to be flattened to less than ⅜ in. high.

4. **Countersunk.**—Where heads less than ⅜ in. high are required, they shall be countersunk. The conventional signs for countersunk rivets mean that rivets shall be countersunk and chipped. Where chipping is not required, it should be so noted on the drawing. Countersunk rivets should be avoided whenever possible.

5. **Clearance of Heads.**—In determining clearance the heights of heads should be assumed as follows:

Full head ⅞ in. rivet	¾ in. high
Full head ¾ in. rivet	⅝ in. high
Full head ⅝ in. rivet	9/16 in. high
Head flattened to ⅜ in. rivet	½ in. high
Countersunk, not chipped	⅛ in. high

6. **Spacing.**—In spacing rivets the use of fractions smaller than ¼ in. should be avoided. Where unavoidable, locate in such a way as to cause the least number of repetitions.

Locate splices and stiffeners with a view to keeping the rivet spacing as regular as possible.

7. **Stagger and Clearance.**—For distances center to center of staggered rivets and clearance required for driving, see standards. In special cases where the prescribed clearances are impossible, allow at least ½ in. clearance for ⅞ in. and 1 in. rivets and 7/16 in. for ¾ in. rivets, from the edge of the rivet head to the nearest surface or other obstruction.

In the connection of cross-frames to girders, and in small lug angles and detail angles, rivets must be spaced so that they will not interfere with each other in driving.

In girder flange angles, the rivets in the "flange" legs should stagger at least 1 in. with rivets in the "web" legs, but should be staggered uniformly.

RIVETED CONNECTIONS.—1. **Grouping.**—Rivets should be grouped to insure that the line of applied stress passes as near as possible through the center of the group of rivets which resists that stress. Where the eccentricity is marked, the stress on the extreme rivet due to this eccentricity shall be computed and when properly combined with the direct stress shall not exceed the allowable stress per rivet.

2. **Gusset Plates.**—Gusset plates shall have such a thickness as will on any section develop, in bending and shear, the full stress which has been transmitted to it by the rivets outside the section.

3. **Clearance.**—The clearance between chords and web members entering same and other similar riveted connections shall be not less than ⅛ in. in heavy structures and 1/16 in. in light structures.

PINS AND PIN PACKING.—1. **Pins.**—Pins shall be proportioned to carry the reactions of the stresses in all the members meeting at a point at unit stresses specified. In computing bending moment on pins, assume each load concentrated at its center of bearing.

2. **Pin Packing.**—Observe the following rules regarding arrangement of eye-bars and pin plates:

(1) Arrange pin packing so as to reduce bending moment on pin to minimum.

(2) Leave at least 1/16 in. clearance between adjacent surfaces.

(3) Provide an additional clearance in the length of the pin of not less than ½ in.

(4) When two or more pin plates are riveted together, allow 1/32 in. for each plate, in addition to its nominal thickness.

(5) Where hinge plates are used allow ⅛ in. clearance between hinge plates and faces of connecting members.

(6) Adjacent surfaces of eye-bars composing a member shall have a clearance of ¾ in. to allow for painting.

(7) All eye-bars are to lie in planes as nearly as possible parallel to the center line of truss, no divergence exceeding one inch in 16 ft. being permitted.

* Prepared by the engineering department of the Chicago, Milwaukee & St. Paul Ry.; Mr. C. F Loweth, Chief Engineer, and Mr. J. H. Prior, Office Engineer.

(8) Where distance between adjacent surfaces is ⅜ in. or more, filler rings shall be provided to prevent lateral motion, but the aggregate length of such filler rings shall be ½ in. less than the neat length required, after making necessary allowances for packing.

(9) The neat grip of pins shall be the distance out to out of outside surfaces after making allowances for clearance.

(10) The ordered length of pins between shoulders shall exceed the neat grip by the following allowances:

For pins of 3½ in. diam. or less, allow ¼ in.
For pins of 3¾ in. diam. to 6 in. diam., allow ½ in.
For pins of 6¼ in. diam. to 9¼ in. diam., allow ¾ in.

GIRDER WEBS.—Width of Web Plates.—On deck girders the web must usually project ¾ in. above the back of the top flange angles, to receive the notches in the track ties, except for concrete deck floors where the slabs rest on a top cover plate. In other cases, where no cover plates are required, the web must be flush with the top flange angles. At the bottom flange in all cases, and at the top flange where cover plates are required, the web may be set back ¼ in.

Web plates shall not be ordered in widths having a fraction of an inch less than ½ in.

Thickness.—Web plates should have a minimum thickness of $\frac{7}{16}$ in. At web splices ¼ in. clearance between ends of web plates shall be allowed.

Web Splices Location.—Web splices for girders, when required, should preferably be placed near the third or quarter points, and never when avoidable at the point of maximum moment.

Size.—Web splices should be of sufficient width to take two lines of rivets through each section of the web spliced. When not under floorbeam connection angles, ⅛ in. clearance may be allowed top and bottom.

Moment Splices.—In addition there should be splice plates on the vertical legs of the flange angles, designed to splice the portion of the web covered by the flange and where thus spliced, the resisting moment on the web may be taken as equivalent to that of ⅛ of its gross area considered as flange section.

Where the splice plates on the flange angles are omitted, the rivets in the flange angles for a distance of one foot either side of the splice may be considered as part of the group of splicing rivets, and account shall be taken of the longitudinal shearing stress on these rivets as well as the stress due to the splice.

Riveting.—The riveting shall, where practicable, be such as to develop the full strength of the web, and shall always be such as to develop the actual moment carried by the web at any point; this being determined by multiplying the total moment on the section by the ratio of ⅛ of the gross web section to the total flange area, including this web equivalent. Splices shall also be designed to carry the total shear on the section due to the assumed loading.

GIRDER FLANGES.—1. Composition.—At least ½ of the area of the flange section should consist of angles, or else the maximum size of the latter be used, and in no case should the center of gravity of the flange come above the flange angles. For location of center of gravity for various types of flange and sizes of material, see Table 88, Part II.

2. Composition of flanges shall preferably be as follows:

(1) 6″ × 6″ angles without cover plates.
(2) 6″ × 6″ angles with 14 in. or 16 in. cover plates.
(3) 8″ × 8″ angles with 17 in. or 18 in. cover plates.
(4) 8″ × 8″ angles with 2 or 4–6″ × 4″ angles, without cover plates. (Type A4.)

Thickness of flanges without cover plates shall not be less than $\frac{1}{12}$ the width of the outstanding leg of the angle.

3. **Net Section.**—The riveting in the tension flanges shall be computed according to method shown in Tables 109 to 113, Part II. Where the spacing of flange rivets is not known in advance, about the following allowances shall be made. In detailing flange riveting, where there is not a considerable excess of flange section, endeavor to keep within these allowances:

(1) Flange angles without cover plates and without lateral bracing connections, each angle—one hole out.

(2) Flange angles without cover plates, but with lateral connections, each angle—1½ holes out.

(3) Flange angles with cover plates, each angle—two holes out.

(4) Cover plates—two holes out.

4. **Cover Plates.**—Cover plates shall have the same thickness or shall diminish in thickness from the flange angle out. In determining length of cover plates, the curve of maximum moments shall be established and plates shall be made 1 ft. longer at each end than the theoretical requirement.

5. **Flange Splices.**—Flanges shall never be spliced unless it is impossible to get material of the required length. Where flange splices occur the following requirements shall be observed:

(1) Splices shall always be located at points where there is an excess of flange section.
(2) No two parts of the flange shall be spliced within 2 ft. of each other.
(3) Flange angles shall be spliced with a splice angle of equal section riveted to both legs of the angle spliced. Where this is impossible, the largest possible splice angle shall be used, and the difference made up by a plate riveted to the vertical leg of the opposite angle.

(4) In splicing cover plates where one or more plates intervene between the splice plate and the cover plate which it splices, the requirement of paragraph 57 of the A. R. E. A. Specifications for Design shall be observed.

(5) Rivets in splice plates and angles shall be located as close together as possible, in order that the transfer may take place in a short distance.

(6) No allowance shall be made for abutting edges of spliced members of the compression flange.

6. **Flange Riveting.**—Rivets connecting flange to the web shall be sufficient to resist at any point the longitudinal shear combined with any load that is applied directly to the flanges. The wheel loads where ties rest directly on the flanges shall be assumed to be distributed over 3 ft.

The pitch of rivets between flange and web at any section may be computed by the formulas:
For through girders, $p = R \cdot d/S$.

For deck girders, $p = \dfrac{R}{\sqrt{\left(\dfrac{W}{36}\right)^2 + \left(\dfrac{S}{d}\right)^2}}$.

p = longitudinal spacing of rivets in inches;
R = value of one rivet in bearing or double shear in pounds;
d = distance center to center of flanges in inches;
S = total maximum shear in pounds at the section, reduced in the ratio of the net area of flange angles and plates to the net area of flange plus ⅛ the gross web section.
W = one wheel load plus 100 per cent impact.

7. **Maximum Spacing.**—Maximum spacing of rivets between flanges and web shall be:

Top flange, deck girders	3½ in.
Top flange, through girders	4½ in.

For convenience in shop work, spacing of rivets in top and bottom flanges shall be exactly alike where possible.

8. **Rivets in Cover Plates.**—Where it is necessary to compute spacing of rivets connecting cover plates to flange angles, the following formula may be used:

$$p = n \cdot R \cdot d/S \times A/a$$

where R = value of one rivet in single shear or bearing;
n = number of rivets on one transverse line through cover plates and flanges;
a = total area of cover plates at section;
A = area of entire flange at section;
S and d, as in section 6, "Flange Riveting."

The pitch as computed by this formula shall be diminished 15 per cent for every cover plate more than one. Rivets in cover plates shall preferably stagger half way with the rivets in the vertical legs of the flange angles. The maximum spacing shall be 6 in.

9. **Circular Ends.**—For through spans with circular ends, the end angles should be spliced near the ends, as the full length angles cannot be handled in making the bends.

Rivets through cover plates on circular ends must be spaced close enough to draw the plates tight against the angles. The smaller the radius, the closer rivets should be spaced.

10. **Overrun of Angles.**—In plate girders whose top flange is composed of four or more angles, about 1 in. should be allowed between the edges of angles to allow for overrun.

11. **Gage in Cover-Plates.**—On girders which are similar, but which have webs of different thickness, the gage in the angles should be left the same and the gage in the cover plate varied to suit the web thickness.

GIRDER STIFFENERS.—Intermediate Stiffeners.—Intermediate stiffeners, except at concentrated load, may be offset, and shall bear tightly against top and bottom flange. The ordered length of offset stiffener angles shall be the finished length plus the thickness of each angle over which it is offset.

Size of Stiffeners.—In general, the minimum size of stiffeners bearings against 6″ × 6″ flange angles shall be 5″ × 3½″ × ⅜″, and against 8″ × 8″ flange angles shall be 6″ × 3½″ × ⅜″.

Field riveted stiffeners at floorbeams of through girders may have ⅛ in. clearance at the top. Fillers under end stiffeners and under concentrated loads must bear on bottom flange, but may have ⅛ in. clearance at top.

Rivets in Stiffeners.—Rivets in stiffener angles may have the maximum spacing, except that:

(a) Rivets in end stiffeners and stiffeners at concentrated loads shall develop the full computed stress in the stiffeners.

(b) Spacing of rivets in end stiffeners, intermediate stiffeners, and web splices shall be identical, except that rivets in any line may be omitted where possible without exceeding the maximum specified pitch, in order to minimize shop work of punching.

Holes for Hand-Hooks.—All stiffeners on deck girders with concrete decks and ballast floors should have holes punched in the outstanding legs for inserting hand-hook to support a person inspecting bridge. Holes should be $\frac{15}{16}$ in. diameter and located 6 in. from top flange on shallow girders and 6 ft. from bottom flange on deep girders. Gage line of hole to be $1\frac{1}{4}$ in. from outer edge of angle.

STRINGERS AND FLOORBEAMS.—1. **Stringers.**—Stringers for through girder spans may be either I-beams or built girders. Where I-beams are used two stringers shall be placed under each rail. Depth of stringers shall depend on available distance from base of rail to "low bridge"; depth shall be preferably $\frac{1}{6}$ to $\frac{1}{8}$, but not less than $\frac{1}{10}$, the panel length.

2. **Floorbeams.**—Depth of floorbeams shall be such as to allow stringers to be framed readily into the web, and not less than $\frac{1}{6}$ of the distance center to center of girders or trusses.

3. **Stringer Connections.**—Stringers shall be riveted to webs of floorbeams with $\frac{3}{4}$ in. connection angles. Connection angles are to be faced to provide uniform bearing against webs of floorbeams. Make stringers $\frac{1}{32}$ in. short at each end for clearance in erecting.

4. **Floorbeams for Through Girders.**—The gusset plates connecting floorbeams to main girders shall, wherever possible, extend to the top of the girder and shall have an angle riveted along the edge, to form an effective stay for the top flange of the main girder, and they shall also form the webs of the end portions of the floorbeams, extending out toward the center as far as the clearance line will allow, and being there spliced to the main web.

5. **Floorbeams for Truss Bridges.**—Floorbeams for truss spans shall preferably be riveted to the vertical posts or hangers, extending the connection angle above the top flange where necessary to secure sufficient rivets. When it is necessary to cut away the lower corner of the floorbeam to clear the chord, special care shall be taken to so reinforce the web as to carry the end shear into the connection angles.

TRUSS AND TOWER MEMBERS.—1. **Top Chord and End-post.**—The top chord and the inclined end-post shall usually consist of two built channels, with a thin cover plate on top and with bottom flanges latticed. The bottom flanges shall be made heavier than the top, in order that the gravity axis may come as close as possible to the center line of the webs.

2. **Verticals and Rigid Tension Members.**—Intermediate posts shall usually consist of two rolled or built channels latticed. Hip verticals and similar members and the two end panels of the bottom chords of single track pin-connected trusses shall be rigid, and may consist either of two rolled or built channels latticed; or of four angles latticed to form an I-section.

3. **Eye-bars.**—Eye-bars shall be used for all bottom chord members and main diagonals that do not require to be stiffened in pin-connected trusses. Dimensions of heads shall be according to manufacturers shop standard. Length of eye-bars shall be given on the drawings, center to center of pin holes, and also back to back of pin holes.

4. **Eccentricity.**—The line of applied force must coincide with the gravity axes of built members or else the member must be designed for combined direct stress and flexure due to the eccentricity of the applied load.

5. **Bending Due to Weight.**—Bending moment in the top chord and end-post due to weight of member may be computed by the approximate formula, $\frac{P}{A} \pm M \cdot c/I$, where P = total direct stress in the member; A = gross area of the section of the member; M = bending moment at the section of the member in in.-lb.; c = distance to extreme fiber; and I = moment of inertia of the section of the member, and the stress from such bending shall be deducted from the average compressive stress allowed by the column formula.

6. **Bending in End-posts.**—In computing stresses in the end-post of through pin-connected trusses, due to wind force, where the end-post consists of two built or rolled channels, if the product of the wind reaction in the top chord times one-half the distance from the foot of the post to the lowest connection of the portal bracing does not exceed the product of the dead load stress in one of the channels composing the end-post times the distance center to center of the bearings of the channels on the pin, the post may be considered fixed-ended and the point of contra-flexure assumed midway between the foot of the post and the lower connection of the portal bracing. Otherwise it must be considered pin-connected. The end-posts of riveted through trusses shall be considered as fixed-ended columns.

7. **Over-run of Angles.**—Where side plates are used on chord sections placed between the flange angles, at least $\frac{1}{4}$ in. clearance should be allowed between the edges of the plate and the angles to allow for over-run of angles.

8. Clearance for Riveting.—When flanges of angles and channels of built members are turned in, $5\frac{1}{2}$ in. opening between edges of angles or channels is required to rivet the tie plates and lacing.

LATERAL AND SWAY BRACING.—1. **Minimum Sizes.**—The minimum size of angles to be used in bracings shall be $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$. Not less than three rivets shall be used in the connection.

2. **Effective Section.**—Where single angles are used for bracing members without lug angles connecting the outstanding leg to the gusset plates, not more than 80 per cent of the net section, if in tension, shall be considered as effective.

Where single angles, used for bracing members, have lug angles connecting their outstanding legs to the gusset plates, and where the center of the group of connecting rivets in the gusset plates fall close to the gravity line of the angle, in plan, 90 per cent of the net section may be considered effective.

3. **Double Diagonal Systems.**—In double diagonal systems the shear due to wind force shall be considered as carried wholly by one diagonal in tension, but the maximum value of $l/r = 120$, specified for bracing members, shall not be exceeded. In assuming "r" the connection of diagonals at their intersection may be considered as offering support against deflection in the plane of the system, but not against deflection perpendicular thereto.

4. **Bending at Connections.**—Connections between bracing members and chords shall be designed to avoid as far as possible any bending stress in main truss members.

5. **Allowance for Draw.**—For diagonal bracing of one or two angles the following draw should be allowed:

For lengths up to 10 ft.	No Allowance.
from 10 to 21 ft.	Allow $\frac{1}{16}$ in.
from 21 to 35 ft.	Allow $\frac{1}{8}$ in.
over 35 ft.	Allow $\frac{3}{16}$ in.

The use of thirty-seconds of an inch should be avoided but the above allowances should not be varied by more than $\frac{1}{32}$ in.

LATERAL BRACING.—1. **Lateral Bracing.**—Lateral bracing shall be in general as follows:

(1) Deck girders and top flanges of stringers 15 ft. long and over; single diagonal system with transverse struts, composed of single angles. Slope of diagonals 45° to 60° with axis of bridge.

(2) Through girders: Double diagonal system of same panel length as floor system, composed of single angles; floorbeams to act as the transverse struts of the system.

(3) Trusses, loaded chord: Double diagonal systems of same panel length as floor systems, composed of single angles, or double angles back to back; floorbeams to act as the transverse struts of the system.

(4) Trusses, unloaded chord: Double diagonal systems of same panel length as floor system with transverse struts at panel points; all composed of two or four angles laced to form a channel or I-section, of depth equal to depth of chords.

2. **Traction Stresses.**—The lateral system in the plane of the loaded chord of truss spans and of through girder spans shall be effectively riveted to the stringers at intersections, and the diagonal shall be designed to transmit the traction for one panel length of track to the panel point; one diagonal for each stringer considered acting in tension.

3. **Clipping Angles for Clearance.**—The vertical leg of laterals should be clipped at the end when there is a possibility that the square corner would interfere in any way with putting in the laterals or riveting up. This is to be particularly looked out for at floorbeam connections of through girder spans and in top laterals of Type A4 girder spans.

4. **Squaring of Holes in Connections.**—Where laterals are riveted to stringers the holes should be squared with the stringers, if possible. At the intersection of diagonals, the holes in splices with two lines of rivets should be squared with lateral and skewed on the splice plate.

5. **Tie Plates and Lacing Symmetrical.**—Where laterals have tie plates or tie plates and lacing bars, they should be detailed symmetrically so that the angles will be identical by turning end for end.

6. **Lateral Plates C3 and C4 Spans.**—The lateral plates of Type C3 and Type C4 girder spans (flanges two angles and cover plates) should not be shop riveted to the girders, as it is impossible to put in floorbeam connection angles when this is done.

TRANSVERSE BRACING.—1. Transverse bracing shall be used as follows:

(1) At intervals of not more than 15 ft. on deck girder spans. Intermediate frames shall be of minimum material. End frames shall be designed to carry to the abutment the total lateral forces acting on the top flange. End frames of skew deck girders shall be placed at the end of the short girder, and at right angles to same. Top and bottom lateral diagonal braces shall be used to stay the end of the long girder.

(2) As spacers for stringers resting on masonry where end floorbeams cannot be used. These frames shall be riveted to girders or truss shoes where practicable.

(3) As spacers for stringers at all expansion points.

(4) At end panel of through truss spans, having vertical truss members. These frames shall be as deep as clearance will permit.

(5) Through truss spans shall have riveted portal braces rigidly connected to the end-posts and top chords. They shall be as deep as clearance will allow, and shall be designed to carry to the abutment the total wind force acting on the top chord.

(6) At panel points of deck truss spans, having vertical members. Intermediate frames shall be designed to carry ½ the panel concentration of wind and centrifugal force to the bottom chord and the end frame shall be designed to carry ½ the total wind and centrifugal force acting on the top chord to the abutment.

Frames for (1), (2) and (3) shall consist of single angle struts, top and bottom and double diagonals. Frames for (4) may consist of knee braces attached to the top lateral struts, but preferably where clearance permits, of light open webbed girder. Portal frames shall consist of open webbed girders, with knee braces connections to inclined posts. Frames for (6) shall consist of double diagonals running between floorbeams and lower lateral struts and composed of two angles back to back, or of two or four angles laced.

2. **Diaphragms for Twin Deck Spans.**—Diaphragms connecting two pairs of twin girders are to be omitted on shallow spans. Where the girders exceed 3 ft. 6 in. in depth, diaphragms shall be added for rigidity They shall be connected to girders with field bolts.

3. **End Cross Frames and Diaphragms.**—In the design and location of end cross frames and diaphragms their shape and position shall be such as to give access to the space between the girders for inspection, painting and the placing of anchor bolts.

APPENDIX I.

PROGRESS IN RAILWAY BRIDGE DESIGN AND SPECIFICATIONS

1924.

PROGRESS IN RAILWAY BRIDGE DESIGN.—When this book was written, 1914, E 50 loading was sufficient for all except a few very heavy engines, see Table II, while a few railways were using an E 60 loading. In ten years the live loads on railway bridges have increased so that E 60 loading is now the standard loading for most main lines, with a few roads using E 70 on certain heavy traffic sections. While the 1920 A.R.E.A. Specifications for Railway Bridges have been adopted by several roads, and give promise of quite general use, very few bridges have as yet been designed under these specifications. For several years both the 1910 and the 1920 A.R.E.A. specifications will be in general use. For the above reasons it has been decided not to rewrite this chapter but to add new material as an appendix. The essential parts of the 1910 A.R.E.A. specifications have been retained, while the 1920 A.R.E.A. specifications as amended to May, 1923, are printed complete with the exception of the specifications for materials.

STANDARD BRIDGE DESIGN.—A standard design for a 200 ft. span single track railway bridge prepared by the American Bridge Company is given in Fig. 41, Fig. 42 and Fig. 43. The stress sheet is shown in Fig. 41, while the design details are shown in Fig. 42 and Fig. 43. This standard design was prepared by the American Bridge Company in 1919, and complies with the A.R.E.A. 1910 Specifications for Railway Bridges, for an E 60 loading. This design represents standard practice. The allowance for impact in the 1910 A.R.E.A. specifications is somewhat greater than in the 1920 A.R.E.A. specifications, and this bridge is therefore probably slightly heavier than an E 60 bridge built under 1920 A.R.E.A. specifications. The impact allowance for a loaded length of 100 ft. is 75 per cent of the live load in both the 1910 and the 1920 A.R.E.A. specifications, while for a loaded length of 200 ft. the impact allowance in the 1910 specifications is 60 per cent, and in the 1920 is 42 per cent of the live load. For a loaded length less than 100 ft. the impact allowance in the 1920 specifications is slightly greater than in the 1910 specifications. The chords and main web members of a 200 ft. span bridge designed under the 1910 A.R.E.A. specifications will be somewhat heavier than the corresponding members designed under the 1920 A.R.E.A. specifications, while the floor system will weigh slightly less.

RAILWAY BRIDGE SPECIFICATIONS IN 1924.—Since the second edition of this book three general specifications for steel railway bridges have been prepared. (1) The American Railway Engineering Association adopted a revised "General Specifications for Steel Railway Bridges" in 1920. These specifications as revised to May, 1923, are printed in the last part of this chapter. (2) A Special Committee on Specifications for Bridge Design and Construction of the American Society of Civil Engineers has prepared "Specifications for Steel Railway Bridge Superstructure." These specifications are printed in Trans. Am. Soc. C. E., Vol. 86, pp. 471 to 531, 1923. (3) The Canadian Engineering Standards Committee has prepared a specification for railway bridges that has been generally adopted by the railways in Canada. These specifications as adopted in 1920 and revised in 1922 agree in all essentials with the A.R.E.A. specifications. The most important features of specifications (1) and (2) will be briefly discussed. Specification (3) is in substantial agreement with specification (1).

Clearances.—Both specifications specify a horizontal clearance of 16 ft., and a vertical clearance of 22 ft. above the top of rail.

Live Loads.—The A.R.E.A. specifications specify a live load of E 60 followed by a train load of 6,000 lb. per lineal ft. or two concentrated axle loads of 75,000 lb. each, axles spaced 7 ft. In special cases E 45 loading may be specified. A detailed study of the relative effect of the various engine loadings in use and the E loading is published in Proceedings Am. Ry. Eng. Assoc., Vol. 21, p. 561, also in Trans. Am. Soc. C. E., Vol. 86, p. 532, 1923.

As a result of this study, the Cooper series was recommended by the bridge committee of the American Railway Engineering Association and adopted by that Association. The reasons presented by the Committee were as follows:

1.—"No one existing type of locomotive loading gives maximum moments and shears for spans of all lengths, whereas the system recommended does approximate the high points of the curves of all the existing types.

2.—"This system of loading produces stresses slightly smaller in short-span bridges and slightly greater in long-span bridges than those produced by the heaviest types in operation, which provides adequately for the engines now in use, and for future development in engine and car loadings which will increase the load per foot on long structures.

3.—"This system is the adopted standard of measurement of strength of existing bridges on the majority of roads, and, having been in use for a number of years, conveys a clear picture to engineers and operating officials."

While several roads, including the New York Central Lines, the Lake Erie and Western, the Santa Fe System and others have adopted an E 70 loading on main lines, bridges designed for E 60 loading will carry all except a very few of the heaviest engines without exceeding the A.R.E.A. unit stresses. The stresses due to the heaviest locomotives in per cent of stresses due to E 60 are given in Table XVII. This table contains all maximum moments and shears except for Engine No. 4, which has a relative moment of 1.03 and a relative shear of 1.01 for a 40 ft. span.

The A.S.C.E. specifications specify a live load of E 60 as above, or a live loading of M 50, for main line bridges. A moment table for a M 10 loading is given in Table XVIII. The loads in the M loading that will produce maximum stresses are given in Table XIX. A conversion table, class E to class M Engine loading, is given in Table XX. The M loading was proposed by Mr. D. B. Steinman in a paper "Locomotive Loadings in Railway Bridges." Trans. Am. Soc. C. E., Vol. 86, pp. 606 to 737, 1923. The equivalent E loadings for M 50 loadings, up to spans of 250 ft. may be calculated from Table XX. The ratings for longer spans are taken from Mr. Steinman's paper and are shown in Table XXI.

TABLE XVII.

HEAVIEST LOCOMOTIVES AND RELATIVE STRESSES FOR SPANS OF 30 FT., 100 FT. AND 200 FT.

From *Trans. Am. Soc. C. E.*, Vol. 86, p. 533, 1923.

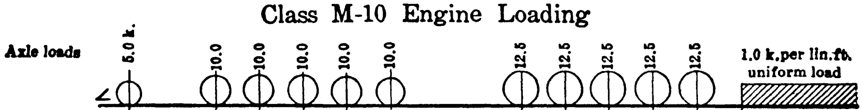
No.	Engine	Proportional Stress for Span					
		30 ft.		100 ft.		200 ft.	
		Moment	Shear	Moment	Shear	Moment	Shear
	E 60	1.00	1.00	1.00	1.00	1.00	1.00
1	2 Engines 4-8-2B	.93	.92	.88	.85	.91	.88
2	2 Engines 4-6-2B	.84	.85	.83	.79	.84	.84
3	1 Engine 2-6-6-2B	.87	.93	.95	.93	.83	.87
4	2 Engines 2-10-2B	1.02	1.00	.96	.90	.91	.94
5	2 N.Y.C. R.R. Engines H-7A2-8-2	.99	.99	.89	.87	.88	.88
6	2 N.Y.C. R.R. Engines 2-8-2	.86	.87	.82	.80	.83	.83
7	2 Santa Fe Type Engines 2-10-2	.92	.89	.94	.88	.98	.94
21	1 Virginian Ry. Engine 2-8-8-2	1.01	1.01	1.19	1.08	.95	.96
22	1 L.S. & M.S. Engine 0-8-8-0	.96	.99	1.00	.99	.82	.90
23	1 Virginian Ry. Engine 2-10-10-2	1.12	1.09	1.23	1.15	1.07	1.09
24	2 Erie Engines 2-10-2	1.16	1.14	1.05	.98	.98	.99
25	1 Erie Engine 2-8-8-2B	.99	.98	1.08	1.02	.91	.94
26	2 Penn. R.R. Engines NIS, 2-10-2	1.27	1.27	1.11	1.01	1.00	1.05

All locomotives followed by 210,000 lb. cars.

TABLE XVIII.

MOMENTS, IN THOUSAND FOOT-POUNDS, FOR CLASS M-10 ENGINE LOADING.

Class M-10 Engine Loading



Wheel Numbers		1	2	3	4	5	6	7	8	9	10	11	
Spacing in feet		10	5	5	5	5	15	5	5	5	5	5	
Summations	Kips	5.0	15.0	25.0	35.0	45.0	55.0	67.5	80.0	92.5	105.0	117.5	117.5
	Feet	0	10	15	20	25	30	45	50	55	60	65	70
	Kips	117.5	112.5	102.5	92.5	82.5	72.5	62.5	50.0	37.5	25.0	12.5	0
	Feet	70	60	55	50	45	40	25	20	15	10	5	0
Moments of Wheel Loads About	End of Train	3787.5	3437.5	2837.5	2287.5	1787.5	1337.5	937.5	625 0	375 0	187.5	62.5	
Wheel Number	11	3200.0	2875 0	2325 0	1825.0	1375 0	975 0	625.0	375 0	187.5	62.5		
	10	2675 0	2375 0	1875 0	1425 0	1025 0	675 0	375 0	187.5	62.5	62.5		
	9	2212.5	1937.5	1487.5	1087.5	737.5	437.5	187.5	62.5	62.5	187.5		
	8	1812.5	1562.5	1162.5	812.5	512.5	262.5	62.5	62.5	187.5	375.0		
	7	1475 0	1250.0	900.0	600 0	350.0	150 0	62.5	187.5	375.0	625.0		
	6	650 0	500 0	300.0	150 0	50 0	187.5	437.5	750 0	1125 0	1562.5		
	5	425.0	300.0	150.0	50 0	50 0	300 0	612.5	987.5	1425 0	1925.0		
	4	250.0	150 0	50 0	50.0	150.0	462.5	837.5	1275.0	1775.0	2337.5		
	3	125.0	50 0	50.0	150 0	300.0	675.0	1112.5	1612.5	2175 0	2800.0		
	2	50.0	50.0	150.0	300.0	500 0	937.5	1437.5	2000.0	2625 0	3312.5		
	1	100 0	250.0	450 0	700.0	1000.0	1562.5	2187.5	2875.0	3625.0	4437.5		

TABLE XIX.

WHEEL DETERMINING MAXIMUM MOMENT FOR CLASS M ENGINE LOADING.

Segments in feet	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	100	110
1000 to 300	2	3	3	3	4	4	5	5	6	6	7	7	8	8	9	9	9	10	10	11
290 to 250	2	3	3	3	4	4	5	5	6	7	7	7	8	8	9	9	9	10	10	11
240	2	3	3	3	4	4	5	6	6	7	7	7	8	8	9	9	9	10	10	11
230	2	3	3	3	4	4	5	6	6	7	7	7	8	8	9	9	9	10	10	11
220	2	3	3	3	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	11
210	2	3	3	3	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	11
200 to 160	2	3	3	3	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10	11
150	2	3	3	3	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10	11
140	2	3	3	4	4	5	5	6	6	7	7	8	8	9	9	9	10	10	10	11
130	2	3	3	4	4	5	5	6	7	7	7	8	8	9	9	9	10	10	10	11
120	2	3	3	4	4	5	5	6	7	7	8	8	8	9	9	9	10	10	10	11
110	2	3	4	4	4	5	5	6	7	7	8	8	8	9	9	9	10	10	10	11
100	2	3	4	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	10	11
95	2	3	4	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	10	
90	$\overline{11}$	3	4	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10		
85	$\overline{11}$	$\overline{11}$	$\overline{11}$	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	7	7	7	8	8	9	9	9	9	10			
80	$\overline{11}$	$\overline{11}$	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	7	8	8	9	9	9	10				
75	$\overline{11}$	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{9}$	8	8	9	9	9					
70	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	8	8	9	9						
65	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	9							
60	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	8								
55	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{8}$	$\overline{8}$									
50	7	$\overline{10}$	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{8}$	$\overline{8}$	$\overline{8}$										
45	7	8	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{8}$	$\overline{8}$	$\overline{8}$											
40	7	8	8	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{8}$	$\overline{8}$												
35	7	8	8	9	$\overline{9}$	$\overline{9}$	$\overline{8}$													
30	7	8	9	9	9	$\overline{9}$														
25	7	8	9	9	9															
20	7	8	9	9																
15	7	8	9																	
10	7	8																		
5	7																			

The shorter segment is ahead followed by the longer one except where the wheel is overlined

Class M Engine

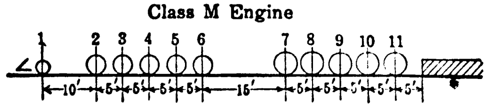
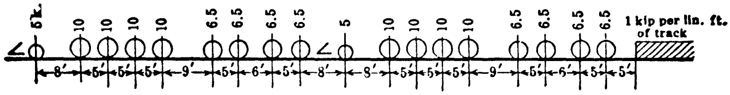


TABLE XX.

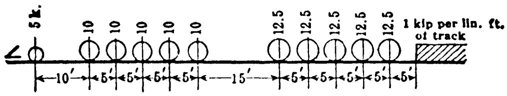
Conversion Table, Class E to Class M Engine Loading.

To Convert E-Rating to Equivalent M-Rating Multiply by the Coefficients in this Table
To Convert M-Rating to Equivalent E-Rating Divide by the Coefficients in this Table

Class E-10 Engine Loading



Class M-10 Engine Loading



Span length, in feet	Maximum moment	Maximum shear	Maximum floor beam reaction	Span length, in feet	Maximum moment	Maximum shear	Maximum floor beam reaction
10	0.800	0.800	0.800	46	0.755	0.771	0.777
11	0.801	0.799	0.769	48	0.759	0.771	0.777
12	0.800	0.800	0.747	50	0.765	0.775	0.778
13	0.800	0.800	0.732	52	0.769	0.772	0.782
14	0.801	0.799	0.730	54	0.769	0.767	0.786
15	0.800	0.800	0.729	56	0.774	0.760	0.793
16	0.800	0.800	0.727	58	0.777	0.759	0.803
17	0.801	0.800	0.725	60	0.779	0.756	0.812
18	0.800	0.800	0.724	62	0.781	0.756	0.818
19	0.800	0.799	0.729	64	0.782	0.759	0.825
20	0.800	0.800	0.739	66	0.785	0.760	0.831
21	0.800	0.786	0.746	68	0.785	0.766	0.837
22	0.786	0.773	0.754	70	0.786	0.770	0.845
23	0.772	0.763	0.757	72	0.784	0.775	0.850
24	0.760	0.760	0.761	74	0.781	0.784	0.857
25	0.751	0.758	0.764	76	0.782	0.790	0.862
26	0.743	0.756	0.765	78	0.778	0.795	0.867
27	0.735	0.752	0.770	80	0.776	0.801	0.873
28	0.731	0.749	0.774	82	0.775	0.808	0.877
29	0.729	0.746	0.778	84	0.775	0.815	0.881
30	0.730	0.748	0.778	86	0.775	0.825	0.885
31	0.729	0.750	0.779	88	0.774	0.828	0.887
32	0.728	0.752	0.782	90	0.774	0.839	0.893
33	0.726	0.751	0.786	92	0.774	0.850	0.895
34	0.726	0.752	0.788	94	0.777	0.854	0.898
35	0.724	0.751	0.789	96	0.777	0.855	0.901
36	0.726	0.754	0.786	98	0.777	0.859	0.906
37	0.728	0.753	0.782	100	0.776	0.858	0.909
38	0.731	0.757	0.781	125	0.823	0.874	0.937
39	0.732	0.758	0.782	150	0.859	0.883	0.953
40	0.738	0.759	0.782	175	0.886	0.893	0.964
42	0.745	0.768	0.779	200	0.905	0.902	0.971
44	0.752	0.768	0.776	250	0.939	0.918	0.981

TABLE XXI.

EQUIVALENT E LOADINGS FOR M 50 LOADINGS.

Span, ft.	E Loading for Maximum Moment	E Loading for End Shear	Span, ft.	E Loading for Maximum Moment	E Loading for End Shear
10	E 62.5	E 62.5	300	E 52.5	E 54.2
50	E 65.4	E 64.5	400	E 51.4	E 53.3
100	E 64.4	E 58.3	500	E 51.0	E 52.7
200	E 55.2	E 55.4	600	E 50.6	E 52.4

Impact.—The A.R.E.A. specification for impact is given in § 28 of the 1920 specifications. The A.S.C.E. specifications require that impact be calculated by the formula

$$I = S\frac{2{,}000 - L}{1{,}600 + 10L},$$

in which,

I = impact or dynamic increment to be added to live load stresses;
S = computed maximum live load stress;
L = loaded length of track, in feet, producing the maximum stress in the member. For bridges carrying more than one track, the aggregate length of all tracks producing the stresses shall be used.

Impact shall not be added to stresses produced by longitudinal and lateral or wind forces.

For bridges designed exclusively for electric traction, impact shall be taken as one-third of that given by the impact formula.

TABLE XXII

COMPARISON OF IMPACT FORMULAS.

Specifications	Impact Ratio for Loaded Length in Feet				
	0	50	100	200	400
1910 A.R.E.A.	1.00	.86	.75	.60	.43
1920 A.R.E.A.	1.00	.92	.75	.42	.16
1923 A.S.C.E.	1.25	.93	.73	.50	.29

The impact ratios for several spans as calculated for the impact allowance specified in the 1910 and 1920 A.R.E.A., and the 1923 A.S.C.E. specifications are given in Table XXII. The A.S.C.E. impact formula gives larger ratios below 100 ft. than either the 1910 or 1920 A.R.E.A. impact formulas, and gives ratios that are practically the mean of these two formulas above 100 ft.

Allowable Stresses.—The A.R.E.A. allowable stresses are given in § 38 of the 1920 specifications.

In the A.S.C.E. specifications the allowable compression on columns is given by the formula

$$p = \frac{16{,}000}{1 + \dfrac{l^2}{13{,}500\, r^2}}$$

in which:

p = allowable unit stress;
l = length of member, in inches;
r = least radius of gyration of member, in inches;

but not to exceed the value for $l/r = 40$.

In the A.S.C.E. specifications the shear in plate girder and I-beam webs, net section is 12,000 lb. per sq. in., in place of 10,000 lb. per sq. in. on gross section in the A.R.E.A. specifications.

Net Sections.—The A.R.E.A. specification for net sections is given in § 77 of the 1920 specifications.

The A.S.C.E. specification for net sections is as follows:

" Net sections shall be used in all cases in calculating tension members; and, in deducting rivet holes, they shall be taken as ⅛ in. larger than the nominal diameter of the rivet. The weakening effect of a staggered rivet shall be allowed for by deducting from the transverse section a strip, w in width, as given by the formula

$$w = h - s^2/4g,$$

in which,

w = width, in inches, of strip to be deducted;

h = diameter of rivet hole, in inches;

s = stagger, or longitudinal spacing of rivet with respect to rivet on last gauge line, in inches;

g = distance between gauge lines, or transverse spacing, in inches."

Specifications for Material.—The A.S.C.E. specifications have adopted the A.S.T.M. specifications for Steel for Railway Bridges, while the 1920 A.R.E.A. specifications have adopted A.S.T.M. specifications with slight modifications as noted.

Adoption of A.R.E.A. Specifications.—The following report of the adoption of the 1920 General Specifications for Steel Railway Bridges is taken from the A.R.E.A. Bulletin, October, 1923.

Out of 92 railways, 205,515 miles, 26 railways, 62,075 miles, have adopted the A.R.E.A. specifications in complete form; 36 other railways, 87,156 miles, have incorporated provisions of the A.R.E.A. specifications in their own specifications, while 17 railways, 21,089 miles, have signified that they will adopt the A.R.E.A. specifications in whole or in part. The A.R.E.A. specifications will soon be adopted in whole or in part on 83 per cent of the railway mileage.

The roads that have adopted the A.R.E.A. specifications complete include the A.T. and S.F., 8,862 miles; C.B. & Q., 9,389 miles; C. & N.W., 8,402 miles; C.M. & St.P., 10,261 miles; Great Northern, 8,266 miles. The roads that find objectionable features that will prevent them from adopting the A.R.E.A. specifications include the Penn. System, 10,531 miles; and the Southern Pacific, 7,118 miles.

GENERAL SPECIFICATIONS FOR STEEL RAILWAY BRIDGES.

American Railway Engineering Association.

For Fixed Spans Less Than 300 Feet in Length

1920

Second Edition—May, 1923

I. PROPOSALS AND DRAWINGS

1. **Definitions of Terms.**—The term " Engineer " refers to the Chief Engineer of the Company or his subordinates in authority. The term " Inspector " refers to the inspector or inspectors representing the Company. The term " Company " refers to the Railway Company or Railroad Company party to the contract. The term " Contractor " refers to the manufacturing or fabricating contractor party to the contract.

2. **Proposals.**—Bidders shall submit proposals to conform with the terms in the letter of invitation. The proposals preferably shall be based upon plans and specifications furnished by the Company showing the general dimensions necessary for designing the structure, the stresses and the general or typical details. Invitations covering work to be designed or erected by the Contractor shall state the general conditions at the site, such as track spacing, character of foundations, old structures, traffic conditions, etc.

3. **Drawings to Govern.**—Where the drawings and the specifications differ, the drawings shall govern.

4. **Patented Devices.**—The Contractor shall protect the Company against claims on account of patented devices or parts proposed by him.

5. **Drawings.**—After the contract has been awarded and before any work is commenced, the Contractor shall submit to the Engineer for approval duplicate prints of stress sheets and shop drawings, unless such drawings shall have been prepared by the Company. The tracings of these drawings shall be the property of and be delivered to the Company after the completion of the contract. Shop drawings shall be made on the dull side of the tracing cloth, 24 by 36 inches in size, including margins. The margin at the left end shall be 1½ inches wide, and the others ½-inch. The title shall be in the lower right-hand corner. No changes shall be made on any approved drawing without the consent, in writing, of the Engineer.

6. The Contractor shall be responsible for the correctness of his drawings, and for shop fits and field connections, although the drawings may have been approved by the Engineer.

7. Any material ordered by the Contractor prior to the approval of the drawings shall be at his risk.

II. GENERAL FEATURES OF DESIGN

8. **Materials Used.**—Structures shall be made wholly of structural steel except where otherwise specified. Cast steel preferably shall be used for shoes and bearings. Cast iron may be used only where specifically authorized by the Engineer.

9. **Types of Bridges.**—The different types of bridges may be used as follows:

Rolled beams for spans up to 35 feet.
Plate girders for spans from 30 feet to 125 feet.
Riveted trusses for spans from 100 feet to 300 feet.
Pin-connected trusses for spans from 150 feet to 300 feet.

10. **Number of Trusses.**—Unless otherwise specified, double-track through bridges shall have only two trusses or girders, and four-track bridges three.

11. **Dimensions for Calculation.**—The dimensions for the calculation of stresses shall be as follows:

Span Length.—For trusses and girders, the distance center to center of end bearings.

For floorbeams, the distance center to center of trusses or girders.

For stringers, the distance center to center of floorbeams.

Depth.—For riveted trusses, the distance between centers of gravity of chord sections.

For pin-connected trusses, the distance center to center of chord pins.

For plate girders, floorbeams and stringers, the distance between centers of gravity of flanges, but not to exceed the distance back to back of the flange angles.

12. **Spacing of Trusses, Girders and Floorbeams.**—The width center to center of girders or trusses shall be not less than one-fifteenth of the effective span, and not less than is necessary to prevent overturning under the assumed lateral loading. Panel lengths shall not exceed $1\frac{1}{2}$ times the width c. to c. of trusses or girders.

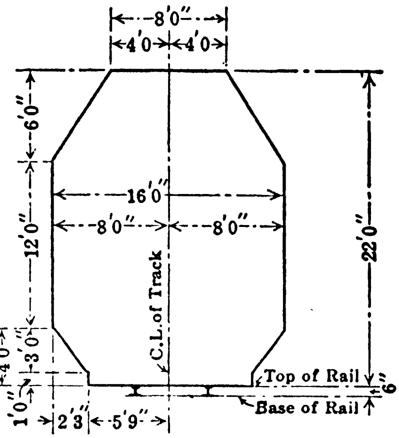


FIG. 1.

13. **Clearances.**—If the alinement is straight, clearances shall be not less than shown on the diagram, Fig. 1. If the alinement is curved, the width of the diagram shall be increased so as to provide the same minimum clearances for a car 80 feet long, 14 feet high and 60 feet center to center of trucks, allowance being made for curvature and superelevation of rails. The height of rail shall be assumed as 6 inches.

14. **Deck Spans on Curves.**—Deck spans on curves shall have the center line of the span placed, usually, so as to bisect the middle ordinate of and be parallel with the chord of the curve.

15. **Skew Bridges.**—In skew bridges without ballasted floors, the ends of stringers or girders for each track shall be square with the track.

16. **Ambiguity of Stress.**—Structures shall be designed so as to avoid, as far as practicable, ambiguity in the determination of the stresses.

III. LOADS

17. **Loads.**—The structures shall be proportioned for the following loads:

a. The dead load.
b. The live load.
c. The impact or dynamic effect of the live load.
d. The lateral loads and forces.
e. The centrifugal force, including impact.
f. The longitudinal force.

Stresses due to these loads and forces shall be shown separately on the stress sheets.

18. Members shall be proportioned for that combination of stresses which gives the maximum total stress, except as otherwise provided.

19. **Dead Load.**—The dead load shall consist of the estimated weight of the entire suspended structure. Timber shall be assumed to weigh $4\frac{1}{2}$ pounds per foot B. M., ballast 120 pounds per cubic foot, reinforced concrete 150 pounds per cubic foot, waterproofing 150 pounds per cubic foot, and rails and fastenings 150 pounds per linear foot of track. If ballast is used, it shall be assumed level with the base of rail and the weight of the ties shall be neglected. Ballasted floors shall have at least 6 inches of ballast under the ties.

20. **Live Load.**—The minimum live load for each track shall be as shown in Figs. 2 and 3, except as modified in Article 21.

The loading that gives the larger stresses shall be used.

21. In special locations, where the conditions limit the loading to light engines, a lighter loading, as stipulated by the Engineer, may be used, but not in any case lighter than three-fourths of that specified in Article 20.

Other live loadings shall be proportional to the loading specified in Article 20, with the same wheel spacing.

22. A train load of 1,200 pounds per linear foot of one track shall be used in determining the stability of spans and towers against overturning.

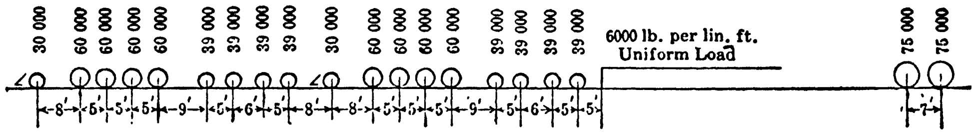


FIG. 2.

23. **Multiple Tracks.**—In calculating the maximum stresses due to live load and centrifugal force when two, three or four tracks are simultaneously loaded, use the following percentages of the specified live load:

For two tracks, loaded, 90 per cent.
For three tracks, loaded, 80 per cent.
For four tracks, loaded, 75 per cent.

24. **Floors.**—Wooden ties shall be designed for the maximum wheel load specified distributed over three ties and with 100 per cent impact added. The fiber stress shall not exceed 2,000 pounds per square inch. The ties shall be not less than 10 feet in length. They shall be placed with openings not to exceed 4 inches in width and shall be secured against bunching. The maximum dap of ties shall be $1\frac{1}{4}$ inches.

25. Floors consisting of beams transverse to the axis of the structure shall be designed for a uniform live load of 15,000 pounds per linear foot for each track, when the minimum live load specified in Article 20 is used. When heavier loadings are used, this uniform load shall be increased proportionately.

26. Floors consisting of longitudinal beams shall be designed for the wheel loads specified.

27. In ballasted floor bridges, the live load shall be considered as uniformly distributed laterally over a width of 10 feet.

28. **Impact.**—The dynamic increment of the live load shall be added to the maximum computed live load stresses and shall be determined by the formula,

$$I = S\frac{300}{300 + \frac{L^2}{100}}, \text{ in which}$$

I = impact or dynamic increment to be added to a live-load stress.
S = computed maximum live-load stress.
L = the length in feet of the portion of the span which is loaded to produce the maximum stress in the member.

29. For bridges designed exclusively for electric traction, the impact stresses shall be taken as one-half of those given by the formula in Article 28.

30. Impact shall not be added to stresses produced by longitudinal or lateral forces, or by the train load specified in Article 22.

31. **Eccentricity of Load on Curves.**—For bridges on curves, provision shall be made for the increased load carried by any truss, girder or stringer due to the eccentricity of the load.

32. **Lateral Forces.**—The wind force on the structure shall be a moving load of 30 pounds per square foot on $1\frac{1}{2}$ times its vertical projection on a plane parallel with its axis, but not less than 200 pounds per linear foot at the loaded chord or flange, and 150 pounds per linear foot at the unloaded chord or flange.

The wind force on the train shall be a moving load of 300 pounds per linear foot on one track, applied 8 feet above the base of rail.

33. The lateral force to provide for the effect of the sway of the engines and train in addition to the wind loads specified in Article 32, shall be a moving load equal to 5 per cent of the specified live load on one track, but not more than 400 pounds per linear foot, applied at the base of rail.

34. The lateral bracing between compression chords or flanges and between the posts of viaduct towers shall be capable or resisting a transverse shear in any panel equal to $2\frac{1}{2}$ per cent of the total axial stress in the chords or posts.

35. In proportioning the bracing, Articles 32 and 33 shall be combined, or Article 34 used alone.

36. **Centrifugal Force.**—On curves, the centrifugal force (assumed to act 6 feet above the rail) shall be taken equal to a percentage of the live load including impact according to the following table:

Degree of Curve	0° 20′	0° 40′	1°	2°	3°	4°	5°	6°	7°	8°	9°	10°	11°	12°
Percentage	$2\frac{1}{2}$	5	$7\frac{1}{2}$	10	10	10	10	10	10	10	10	10	10	10
Speed in miles per hour	80	80	80	65	53	46	41	38	35	33	31	29	28	27

37. **Longitudinal Force.**—Provision shall be made in the design for the effect of a longitudinal force of 20 per cent of the live load on one track only, applied 6 feet above the top of the rail. In structures (such as ballasted deck bridges of only three or four spans) where, by reason of continuity of members or frictional resistance, the longitudinal force will be largely directed to the abutments, its effect on the superstructure shall be taken as one-half that specified above.

IV. UNIT STRESSES AND PROPORTIONING OF PARTS

38. The several parts of structures shall be so proportioned that the unit stresses will not exceed the following, except as modified in Articles 46 and 47:

	Pounds per sq. in.
Axial tension, net section	16,000
Axial compression, gross section	$15{,}000 - 50l/r$
but not to exceed	12,500
l = the length of the member in inches.	
r = the least radius of gyration of the member in inches.	
Tension in extreme fibers of rolled shapes, built sections and girders, net section	16,000
Tension in extreme fibers of pins	24,000
Shear in plate girder webs, gross section	10,000
Shear in power-driven rivets and pins	12,000
Bearing on power-driven rivets, pins, outstanding legs of stiffener angles, and other steel parts in contact	24,000

The above-mentioned values for shear and bearing shall be reduced 25 per cent for countersunk rivets, hand-driven rivets, floor-connection rivets, and turned bolts.

Bearing on expansion rollers, per linear inch	$600d$
d = the diameter of rollers in inches.	

	Pounds per sq. in.
Bearing on granite masonry	800
Bearing on sandstone and limestone masonry	400
Bearing on concrete masonry	600

39. For cast steel in shoes and bearings, the above mentioned unit stresses shall apply.

40. The diagonal tension in webs of girders and rolled beams at sections where maximum shear and bending occur simultaneously, shall not exceed 16,000 pounds per square inch.

41. **Effective Bearing Area.**—The effective bearing area of a pin, a bolt or a rivet shall be its diameter multiplied by the thickness of the piece, except that for countersunk rivets, half the depth of the countersink shall be omitted.

42. **Effective Diameter of Rivets.**—In proportioning rivets, the nominal diameter of the rivet shall be used.

43. **Proportioning Web Members.**—Web members shall be so proportioned than an increase of live load which will increase the total unit stresses in the chords 50 per cent will not produce total unit stresses in the web members more than 50 per cent greater than the designing stresses.

44. **Reversal of Stress.**—Members subject to reversal of stress under the passage of the live load shall be proportioned as follows:

Determine the resultant tensile stress and the resultant compressive stress and increase each by 50 per cent of the smaller; then proportion the member so that it will be capable of resisting either increased resultant stress. The connections shall be proportioned for the sum of the resultant stresses.

45. **Combined Stresses.**—Members subject to both axial and bending stresses (including bending due to floor beam deflection) shall be proportioned so that the combined fiber stresses

will not exceed the allowed axial stress. In members continuous over panel points, only three-fourths of the bending stress computed as for simple beams shall be added to the axial stress.

46. Members subject to stresses produced by a combination of dead load, live load, impact and centrifugal force, with either lateral or longitudinal forces, or bending due to lateral action, may be proportioned for unit stresses 25 per cent greater than those specified in Article 38; but the section shall not be less than that required for dead load, live load, impact and centrifugal force.

47. **Secondary Stresses.**—Designing and detailing shall be done so as to avoid secondary stresses as far as possible. In ordinary trusses without sub-panelling, no account usually need be taken of the secondary stresses in any member whose width measured in the plane of the truss is less than one-tenth of its length. Where this ratio is exceeded, or where subpanelling is used, secondary stresses due to deflection of the truss shall be computed. The unit stresses specified in Article 38 may be increased one-third for a combination of the secondary stresses with the other stresses, but the section shall not be less than that required when secondary stresses are not considered.

48. **Compression Flanges.**—The gross area of the compression flanges of plate girders and rolled beams shall not be less than the gross area of the tension flanges, but the stress per square inch of gross area shall not exceed

$$16{,}000 - 150l/b, \text{ in which}$$

l = the length of the unsupported flange, between lateral connections or knee braces.
b = the flange width.

V. DETAILS OF DESIGN

49. **Slenderness Ratios.**—The ratio of length to least radius of gyration shall not exceed

100 for main compression members.
120 for wind and sway bracing.
140 for single lacing, and for double lacing not riveted at intersections.
170 for double lacing riveted at intersections.

50. The lengths of riveted tension members shall not exceed 200 times their least radius of gyration.

51. **Depth Ratios.**—The depth of trusses preferably shall be not less than one-tenth of the span. The depth of plate girders preferably shall be not less than one-twelfth of the span. The depth of rolled beams used as girders and the depth of solid floors preferably shall be not less than one-fifteenth of the span. If less depths than these are used, the section must be increased so that the maximum deflection will not be greater than if these limiting ratios had not been exceeded.

52. **Parts Accessible.**—Details shall be designed so that all parts will be accessible for inspection, cleaning and painting. Closed sections shall be avoided wherever possible.

53. **Pockets.**—Pockets or depressions which would hold water shall have efficient drain holes, or shall be filled with concrete.

54. **Eccentric Connections.**—Members shall be connected so that their gravity axes will intersect in a point. Eccentric connections shall be avoided if practicable, but, if unavoidable, the members shall be proportioned so that the combined fiber stress will not exceed the allowed axial stress.

55. **Effective Area of Angles.**—The effective area of single angles in tension shall be assumed as the net area of the connected leg plus 50 per cent of the area of the unconnected leg. Single angles connected by lug angles shall be considered as connected by one leg.

56. **Counters.**—If web members are subject to reversal of stress, their end connections preferably shall be riveted. Adjustable counters shall have open turnbuckles.

57. **Strength of Connections.**—Connections shall have a strength at least equal to that of the members connected, regardless of the computed stress. Connections shall be made, as nearly as practicable, symmetrical about the axis of the members.

58. **Limiting Thickness of Metal.**—Metal shall not be less than ⅜-inch thick, except for fillers. Metal subject to marked corrosive influences shall be increased in thickness or protected against such influences.

59. **Sizes of Rivets.**—Rivets shall be ¾ inch, ⅞ inch or 1 inch in diameter as specified.

60. **Pitch of Rivets.**—The minimum distance between centers of rivet holes shall be three diameters of the rivet, but the distance preferably shall be not less than 3½ inches for 1 inch rivets, 3 inches for ⅞-inch rivets and 2½ inches for ¾-inch rivets. The maximum pitch in the line of stress for members composed of plates and shapes shall be 7 inches for 1 inch rivets, 6 inches for ⅞-inch rivets and 5 inches for ¾-inch rivets. For angles with two gage lines and rivets staggered, the maximum pitch in each line shall be twice the amounts given above. If two or more web plates are used in contact, stitch rivets shall be provided to make them act in unison.

In compression members, the stitch rivets shall be spaced not more than 24 times the thickness of the thinnest plate in the direction perpendicular to the line of stress, and not more than 12 times the thickness of the thinnest plate in the line of stress. In tension members, the stitch rivets shall be not more than 24 times the thickness of the thinnest outer plate in either direction. In tension members composed of two angles in contact, a pitch of 12 inches may be used for riveting the angles together.

61. **Edge Distance.**—The minimum distance from the center of any rivet hole to a sheared edge shall be: $1\frac{3}{4}$ inches for 1 inch rivets, $1\frac{1}{2}$ inches for $\frac{7}{8}$-inch rivets and $1\frac{1}{4}$ inches for $\frac{3}{4}$-inch rivets; to a rolled edge $1\frac{1}{2}$ inches $1\frac{1}{4}$ inches and $1\frac{1}{8}$ inches, respectively. The maximum distance from any edge shall be eight times the thickness of the plate, but shall not exceed 6 inches.

62. **Size of Rivets in Angles.**—The diameter of the rivets in any angle whose size is determined by calculated stress shall not exceed one-fourth of the width of the leg in which they are driven. In angles whose size is not so determined 1 inch rivets may be used in $3\frac{1}{2}$ inch legs, $\frac{7}{8}$-inch rivets in 3 inch legs, and $\frac{3}{4}$-inch rivets in $2\frac{1}{2}$ inch legs.

63. **Long Rivets.**—Rivets which carry calculated stress and whose grip exceeds four and one-half diameters shall be increased in number at least one per cent for each additional 1/16-inch of grip. If the grip exceeds six times the diameter of the rivet, specially designed rivets shall be used.

64. **Pitch of Rivets at Ends.**—The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivet for a distance equal to one and one-half times the maximum width of the member.

65. **Compression Members.**—In built compression members, the metal shall be concentrated in the webs and flanges. The thickness of each web shall be not less than one-thirtieth of the distance between the lines of rivets connecting it to the flanges. The thickness of cover plates shall be not less than one-fortieth of the distance between the nearest rivet lines.

66. **Outstanding Legs of Angles.**—The width of the outstanding legs of angles in compression (except when reinforced by plates) shall not exceed the following:

a. For stringer flange angles, ten times the thickness.

b. For main members carrying axial stress, twelve times the thickness.

c. For bracing and other secondary members, fourteen times the thickness.

67. **Stay Plates.**—The open sides of compression members shall be provided with lacing bars and shall have stay plates as near each end as practicable. Stay plates shall be provided at intermediate points where the lacing is interrupted. In main members, the length of the end stay plates shall be not less than $1\frac{1}{4}$ times the distance between the lines of rivets connecting them to the outer flanges, and the length of intermediate stay plates shall be not less than three-quarters of that distance. Their thickness shall be not less than one-fiftieth of the same distance.

68. Tension members composed of shapes shall have their separate segments stayed together. The stay plates shall have a length not less than two-thirds of the lengths specified for stay plates on compression members.

69. **Lacing.**—The lacing of compression members shall be proportioned to resist a shearing stress of $2\frac{1}{2}$ per cent of the direct stress. The section shall be made as required by Articles 38 and 49, in which l shall be taken as the distance between connections of the lacing to the main sections..

The minimum width of lacing bars shall be 3 inches for 1-inch rivets, $2\frac{3}{4}$ inches for $\frac{7}{8}$-inch rivets, $2\frac{1}{2}$ inches for $\frac{3}{4}$-inch rivets, and 2 inches for $\frac{5}{8}$-inch rivets.

70. In members composed of side segments and a cover plate, with the open side laced, one-half the shear shall be considered as taken by the lacing. Where double lacing is used, the shear in the plane of the lacing shall be equally distributed between the two systems.

71. Lacing bars of compression members shall be so spaced that the l/r of the portion of the flange included between their connections will be not greater than 40, and not greater than two-thirds of the l/r of the member.

72. In connecting lacing bars to flanges, $\frac{5}{8}$-inch rivets shall be used for flanges less than $2\frac{1}{2}$ inches wide, $\frac{3}{4}$-inch rivets for flanges from $2\frac{1}{2}$ to $3\frac{1}{2}$ inches wide, and $\frac{7}{8}$-inch rivets for flanges $3\frac{1}{2}$ or more inches wide. Lacing bars with at least two rivets in each end shall be used for flanges over 5 inches wide.

73. The angle of lacing bars with the axis of the member shall be not less than 45 degrees for double lacing, and 60 degrees for single lacing. If the distance between rivet lines in the flanges is more than 15 inches and a single-rivet bar is used, the lacing shall be double and riveted at the intersections.

74. **Splices.**—Abutting joints in compression members faced for bearing shall have component parts spliced. The gross area of the splice material shall be not less than 50 per cent of the gross area of the smaller member. In determining the number of rivets in compression splices, the stress in the splice material shall be taken as 15,000 lb. per square inch of gross area.

75. Joints in riveted work not faced for bearing, whether in tension or compression, shall be fully spliced.

76. **Net Section at Pins.**—In pin connected riveted tension members, the net section across the pin hole shall be not less than 140 per cent and the net section back of the pin hole not less than 100 per cent of the net section of the body of the member, and there shall be sufficient rivets to make the material effective.

77. **Net Section Defined.**—The net section of riveted members shall be the least area which can be obtained by deducting from the gross sectional area the areas of holes cut by any plane perpendicular to the axis of the member and parts of the areas of other holes on one side of the plane within a distance of four inches, which are on gage lines one inch or more from those of the holes cut by the plane, the parts being determined by the formula:

$$A(1 - P/4), \text{ in which}$$

A = the area of the hole, and P = the distance in inches of the center of the hole from the plane.

78. In determining the net section, the diameter of the rivet hole shall be taken one-eighth-inch larger than the nominal diameter of the rivet.

79. **Pin Plates.**—Where necessary to give the required section or bearing area, pin holes shall be reinforced on each segment by plates, one of which on each side must be as wide as the outstanding flanges will permit. These plates shall contain enough rivets and be so connected as to transmit and distribute the bearing pressure uniformly over the full cross section and to reduce the eccentricity of the segment to a minimum. At least one full-width plate on each segment shall extend to the far edge of the stay plate and the others not less than 6 inches beyond the near edge.

80. **Indirect Splices.**—If splice plates are not in direct contact with the parts which they connect, rivets shall be used on each side of the joint in excess of the number required in the case of direct contact to the extent of two extra lines for each intervening plate.

81. **Fillers.**—Where rivets carrying stress pass through fillers, the fillers shall be extended beyond the connected member and the extension secured by additional rivets sufficient to develop the value of the filler.

82. **Forked Ends.**—Forked ends on compression members will be permitted only where unavoidable. Where forked ends are used, a sufficient number of pin plates shall be provided to make the jaws of twice the sectional area of the member and they shall be extended as far as necessary in order to carry the stress of the main member into the jaws, but shall not be shorter than required by Article 79.

83. **Pins.**—Pins shall be long enough to secure a full bearing of all parts connected upon the turned body of the pin. They shall be secured by chambered nuts or by solid nuts with washers. Where the pins are bored, through rods with cap washers may be used. The screw ends shall be long enough to admit of burring the threads.

84. Pin connected members shall be held against lateral movement on the pins.

85. **Bolts.**—Where members are connected by bolts, the turned bodies of the bolts shall be long enough to extend through the metal. A washer at least ¼-inch thick shall be used under the nut. Bolts shall not be used except by special permission.

86. **Upset Ends.**—Bars with screw ends shall be upset so that the area at the root of the thread will be at least 15 per cent larger than in the body of the bar.

87. **Sleeve Nuts.**—Sleeve nuts shall not be used.

88. **Expansion.**—Provision shall be made for expansion and contraction at the rate of one inch for every 100 feet in length. The expansion ends shall be secured against lateral movement. In spans more than 250 feet in length, provision shall be made for expansion in the floor.

89. **Expansion Bearings.**—Spans more than 70 feet in length shall have rollers at one end. Spans of less length shall be arranged to slide on smooth surfaces.

90. **Fixed Bearings.**—Bearings and ends of spans shall be secured against lateral motion.

91. **Rollers.**—Expansion rollers shall be not less than 6 inches in diameter. They shall be coupled together with substantial side bars, which shall be so arranged that the rollers can be cleaned readily. Rollers shall be geared to the upper and lower plates.

92. **Pedestals and Shoes.**—Pedestals and shoes preferably shall be made of cast steel. The difference between the top and bottom bearing widths shall not exceed twice the depth. For hinged bearings, the depth shall be measured from the center of the pin. Where built pedestals and shoes are used, the web plates and the angles connecting them to the base plate shall be not less than ¾-inch thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely. The minimum thickness of the metal in cast steel pedestals shall be one inch. Pedestals and shoes shall be so constructed that the load will be distributed uniformly over the entire bearing. Spans more than 70 feet in length shall have hinged bearings at each end.

93. **Inclined Bearings.**—For spans on an inclined grade and without hinged bearings, the sole or masonry plates shall be beveled so that the masonry surfaces will be level.

94. **Name Plates.**—There shall be a name plate, showing in raised letters and figures the name of the manufacturer and the year of construction, bolted to the bridge near each end at a point convenient for inspection.

VI. FLOORS

95. **Types of Floors.**—Floors may consist of steel floorbeams and stringers, with timber cross-ties supporting the rails, or of one of the solid floor types.

96. **Floor Members.**—Floor members shall be designed with special reference to stiffness.

97. Specifications for plate girders shall apply to floorbeams and stringers.

98. **Spacing of Stringers.**—Stringers usually shall be spaced 6 feet 6 inches center to center. If four stringers are used under one track, each pair shall be spaced symmetrically about the rail.

99. **I-Beam Girders.**—Rolled beams supporting timber decks shall be arranged with not more than four, and preferably not less than two beams under each rail. The beams in each group shall be placed symmetrically about the rail, and shall be spaced sufficiently far apart to permit cleaning and painting. They shall be connected by solid web diaphragms near the ends and at intermediate points, spaced not over twelve times the flange width. Bearing plates shall be continuous under each group of beams. End stiffeners shall be used if required by the provisions of § 38.

100. **Floorbeam Connections.**—Floorbeams preferably shall be square to the girders or trusses. They shall be riveted directly to the girders or between the posts of through and deck truss spans.

101. **End Connection Angles.**—The legs of stringer connection angles shall be not less than 4 inches in width, and not less than $\frac{5}{8}$-inch in thickness before facing. Shelf angles shall be provided to support the stringers during erection, but the connection angles shall be sufficient to carry the whole load. Stringers in through spans shall be riveted between the floorbeams.

102. **Stringer Frames.**—Where two lines of stringers are used under each track in panels more than 20 feet in length, they shall be connected by cross frames.

103. **Solid Floor Connections.**—Solid floors shall be connected to the girders or trusses by angles not less than $\frac{5}{8}$-inch thick if to be faced, or $\frac{1}{2}$-inch thick if not to be faced; one angle on each side of the web of I-beams and one on each of the vertical members of troughs, § 223.

104. **Proportioning Solid Floors.**—Solid floors shall be proportioned by the moments of inertia of the sections, using the net sections including the compression side.

VII. BRACING

105. **Design of Bracing.**—Lateral, longitudinal and transverse bracing shall be composed of shapes with riveted connections. Lateral bracing shall have concentric connections to chords at end joints, and preferably throughout. The connections between the lateral bracing and the chords shall be designed to avoid, as far as practicable, any bending stress in the truss members.

106. When a double system of bracing is used, both systems may be considered simultaneously effective if the members meet the requirements, both as tension and compression members.

107. **Lateral Bracing.**—Bottom lateral bracing shall be provided in all bridges except deck plate girder spans less than 50 feet long, from which it may be omitted. Continuous steel or concrete floors will be considered lateral bracing.

108. Top lateral bracing shall be provided in deck spans and in through spans having sufficient head room.

109. **Portal and Sway Bracing.**—Deck truss spans shall have sway bracing at each panel point. The top lateral loads preferably shall be carried to the supports by means of a complete top lateral system, or the loads may be considered as transferred to the bottom lateral system at each sway frame.

110. Through truss spans shall have portal bracing, with knee braces, as deep as the specified clearance will allow.

111. Through truss spans shall have sway bracing at each intermediate panel point if the height of the trusses is such as to permit of a depth of 6 feet or more for the bracing. When the height of the trusses will not permit of such depth, the top lateral struts shall be of the same depth as the chord and shall have knee braces.

112. **Cross-Frames.**—Deck plate girder spans shall be provided with cross-frames at each end proportioned to resist centrifugal and lateral forces, and shall have intermediate cross-frames at intervals not exceeding 18 feet.

113. **Laterals.**—The smallest angle to be used in lateral bracing shall be $3\frac{1}{2}$ by 3 by $\frac{3}{8}$ inches. There shall be not less than three rivets at each end connection of the angles. Angles shall be connected at their intersections by plates.

114. **Clearance.**—Lateral bracing beneath the track shall be low enough to clear the ties.

VIII. PLATE GIRDERS

115. **Spacing of Girders.**—The girders of deck bridges usually shall be spaced 6 feet 6 inches between centers, except that:

a. In single-track deck spans 75 or more feet in length, the girders shall be spaced in accordance with paragraph 12, but not less than 7 feet 6 inches between centers.

b. In bridges on curves, the girders shall be spaced as shown on the plans.

116. **Design of Plate Girders.**—Plate girders shall be proportioned either by the moment of inertia of their net section including compression side; or by assuming that the flanges are concentrated at their centers of gravity. In the latter case, one-eighth of the gross section of the web, if properly spliced, may be used as flange section. For girders having unusual sections, the moment of inertia method shall be used.

117. **Flange Sections.**—The flange angles shall form as large a part of the area of the flange as practicable. Side plates shall not be used except when flange angles exceeding one inch in thickness otherwise would be required.

118. Flange plates shall be equal in thickness, or shall diminish in thickness from the flange angles outward. No plate shall have a thickness greater than that of the flange angles.

119. Where flange cover plates are used, one cover plate of the top flange shall extend the full length of the girder. Other flange plates shall extend at least 18 inches beyond the theoretical end.

120. **Thickness of Web Plates.**—The thickness of web plates shall be not less than $\sqrt{D}/20$, where "D" represents the distance between flanges in inches.

121. **Flange Rivets.**—The flanges of plate girders shall be connected to the web with a sufficient number of rivets to transfer to the flange section the horizontal shear at any point combined with any load that is applied directly on the flange. One wheel load, where ties rest on the flange, shall be assumed to be distributed over 3 feet.

122. **Flange Splices.**—Splices in flange members shall not be used except by special permission of the Engineer. Two members shall not be spliced at the same cross-section and, if practicable, splices shall be located at points where there is an excess of section. The net section of the splice shall exceed by 10 per cent the net section of the member spliced. Flange angle splices shall consist of two angles, one on each side.

123. **Web Splices.**—Web plates shall be symmetrically spliced by plates on each side. The splice plates for shear shall be of the full depth of the girders between flanges. The splice shall be equal to the web in strength in both shear and moment. There shall be not less than two rows of rivets on each side of the joint.

124. **End Stiffeners.**—Plate girders shall have stiffener angles over end bearings, the outstanding legs of which will extend as nearly as practicable to the outer edge of the flange angles. These end stiffeners shall be proportioned for bearing of the outstanding legs on the flange angles, and shall be arranged to transmit the end reaction to the pedestals or distribute it over the masonry bearings. They shall be connected to the web by enough rivets to transmit the reaction. End stiffeners shall not be crimped.

125. **Intermediate Stiffeners.**—The webs of plate girders shall be stiffened by angles at intervals not greater than:

(a) Six feet.
(b) The depth of the web.
(c) The distance given by the formula, $d = t(12{,}000 - S)/40$.

d = the distance between rivet lines of stiffeners in inches.
t = the thickness of the web in inches.
S = web shear in pounds per square inch at the point considered.

126. If the depth of the web between the flange angles or side plates is less than 50 times the thickness of the web, intermediate stiffeners may be omitted.

127. Stiffener angles shall be placed at points of concentrated loading. Such angles shall not be crimped.

128. Intermediate stiffeners shall be riveted in pairs to the web of the girder. The outstanding leg of each angle shall not be less than 2 inches plus one-thirtieth of the depth of the girder, nor more than 16 times its thickness.

129. **Gusset Plates in Through Girders.**—In through plate girder spans, the top flanges shall be braced by means of gusset plates or knee braces with solid webs connected to the floorbeams and extending usually to the clearance line. If the unsupported length of the inclined edge of the gusset plate exceeds 18 inches, the gusset plate shall have one or two stiffening angles riveted along its edge. The gusset plate shall be riveted to a stiffener angle on the girder. Preferably it shall form no part of the floorbeam web.

130. In through plate girder spans with solid floors, there shall be knee-braces with ⅜-inch webs, extending usually to the clearance line, at intervals of about 12 feet. Each knee-brace shall be well riveted to the floor and the girder, especially at the top, and shall have its edge reinforced by one or two angles.

131. **Ends of Through Girders.**—If through plate girders project two feet or more above the base of the rail, the upper corners shall be rounded. In multiple span bridges, usually only the extreme ends shall be rounded. Exposed ends of through girders shall be neatly finished with end plates.

132. **Spans Shipped Riveted.**—Deck plate girder spans less than 50 feet in length shall be shipped riveted complete, unless otherwise specified.

133. **Masonry Bearings.**—End bearings on masonry preferably shall be raised above the coping by metal pedestals.

134. Sole plates shall be not less than $\frac{3}{4}$-inch thick and not less in thickness than the flange plus $\frac{1}{8}$-inch. Preferably they shall not be longer than 18 inches.

135. **Anchor Bolts.**—Anchor bolts shall be $1\frac{1}{4}$ inches in diameter and shall extend 12 inches into the masonry. There shall be washers under the nuts. Anchor bolt holes in pedestals and sole plates shall be $1\frac{5}{8}$ inches in diameter, except that at expansion points the holes in the sole plates shall be slotted.

IX. TRUSSES

136. **Type of Truss and Sections of Members.**—Trusses shall have single intersection web systems and, preferably, inclined end posts. The top chords and end posts shall be made usually of two side segments with one cover plate and with stay plates and lacing on the open side. The bottom chords of riveted trusses shall be symmetrically made, usually of vertical side plates with flange angles. Web members shall be made of symmetrical sections.

137. **Camber.**—The length of members of truss spans shall be such that the camber will be equal to the deflection produced by the combined dead and live loads without impact.

138. **Riveted Members in Pin-Connected Trusses.**—In pin-connected trusses, hip verticals (and members performing similar functions) and, in single track spans, the two panels at each end of the bottom chords shall be riveted members.

139. **Eye-bars.**—The cross sectional area of the head through the center of the pin hole shall exceed that of the body of the eye-bar by at least $37\frac{1}{2}$ per cent. The thickness of the bar shall be not less than one-eighth of the width nor less than one inch, and not greater than 2 inches. The form of the head shall be submitted to the Engineer for approval before the bars are made. The diameter of the pin shall be not less than seven-eighths of the width of the widest bar attached.

140. **Packing.**—The eye-bars of a set shall be packed symmetrically about the plane of the truss and as nearly parallel as practicable, but in no case shall the inclination of any bar to the plane of the truss exceed 1/16-inch per foot. They shall be packed as closely as practicable. They shall be held against lateral movement, and arranged so that adjacent bars in the same panel will not be in contact.

141. **Gusset Plates.**—The thickness of gusset plates connecting the chords and web members of the truss shall be proportionate to the stress to be transferred, but shall not be less than $\frac{1}{2}$-inch.

142. **Facilities for Lifting Span.**—Provision shall be made for lifting the span at the ends.

143. **Masonry Plates.**—Masonry plates shall not be less than one inch thick.

X. VIADUCTS

144. **Type of Viaduct.**—Viaducts shall consist usually of alternate tower spans and free spans of plate girders or riveted trusses supported on bents. The tower spans usually shall be not less than 30 feet long.

145. **Bents and Towers.**—Viaduct bents shall be composed preferably of two supporting columns, and the bents usually shall be united in pairs to form towers. Horizontal diagonal bracing shall be placed in all towers having more than two vertical panels at alternate intermediate panel points. In double track towers, provision shall be made for the transmission of the longitudinal force to both sides.

146. **Single Bents.**—Where long spans are supported on short single bents, such bents shall have hinged ends, or else have their columns and anchorages proportioned to resist the bending stresses produced by changes in temperature.

147. **Bottom Struts.**—The bottom struts of viaduct towers shall be proportioned for the calculated stresses, but in no case for less than one-fourth of the dead load reaction on one pedestal, considered as compressive stress. Provision shall be made in the column bearings for expansion of the tower bracing.

148. **Batter.**—The columns usually shall have a batter transversely of one horizontal to six vertical for single track viaducts, or one horizontal to eight vertical for double track viaducts.

149. **Depth of Girders.**—The depths of girders in viaducts preferably shall be uniform.

150. **Spacing of Girders.**—In single track viaducts, the girder spacing usually shall be uniform throughout, and shall be determined by the spacing for the longest span in the viaduct, according to the rules specified for deck plate girder spans.

151. In double track viaducts, the girders under each track usually shall be spaced 6 feet 6 inches between centers, and the inner lines of girders shall be supported by cross-girders framed between and riveted to the posts.

152. **Girder Connections and Bracing.**—Girders of tower spans shall be fastened at each end to the tops of the posts or cross-girders. Girders between towers shall have one end riveted, and shall be provided with an effective expansion joint at the other end. No bracing or sway frame shall be common to abutting spans.

153. If neither of the girders under a track rests directly over a tower post, bracing shall be provided to carry the longitudinal force into the tower bracing without producing lateral bending stress in the cross-girders or posts.

154. **Sole and Masonry Plates.**—Sole and masonry plates shall be not less than ¾-inch thick.

155. **Anchorage for Towers.**—Anchor bolts for viaduct towers and similar structures shall be designed to engage a mass of masonry the weight of which is at least one and one-half times the uplift.

XI. MATERIALS

(Sections 156 to 205 inclusive conform to the A.S.T.M. Standard Specifications for Structural Steel for Bridges, 1916 edition, Standard Specifications for Steel Castings, 1916 edition, and Standard Specifications for Wrought Iron Bars, 1918 edition, except as to yield point requirements in which minimum yield point is 30,000 and 25,000 lb. per sq. in. in structural steel and rivet steel, respectively, § 178 and § 179, and a footnote to Table II. These sections and the footnote to Table II follow.)

178. **Character of Fracture.**—Test specimens of structural or rivet steel shall show a fracture of uniform silky or bluish gray appearance, entirely free from visible slag inclusions or other foreign substances.

179. **Surface Defects.**—Finished rolled material shall be free from cracks, flaws, injurious seams, blisters, ragged and imperfect edges, and other surface defects. It shall have a smooth finish, and shall be straightened in the mill before shipment.

Note to Table II.—The weight of individual plates ordered to thickness shall not exceed the nominal weight by more than one and one-third times the amount given in this table.

XII. WORKMANSHIP

206. **Class of Work.**—The work shall be " Punched Work " or " Reamed Work " as stipulated.

207. **General.**—The workmanship and finish shall be equal to the best general practice in modern bridge shops. Material at the shops shall be kept clean and protected from the weather as far as practicable.

208. **Straightening Material.**—Rolled material, before being laid off or worked, must be straight. If straightening or flattening is necessary, it shall be done by methods that will not injure the material. Sharp kinks and bends may be cause for rejection.

209. **Finish.**—Shearing and chipping shall be neatly and accurately done and all portions of the work exposed to view shall be neatly finished.

210. **Punched Work.**—In punched work, holes in material whose thickness is not greater than the diameter of the rivets plus ⅛-inch, may be punched full size. Holes in material of greater thickness shall be drilled.

211. **Reamed Work.**—In reamed work, holes in material ⅞-inch thick and less, used for lateral, longitudinal and sway bracing, lacing, stay plates and diaphragms, may be punched full size.

212. Holes in other material ¾-inch thick and less, shall be sub-punched and reamed.

213. Holes in other material more than ¾-inch thick shall be drilled.

214. **Punched Holes.**—Full size punched holes shall be 1/16-inch larger than the nominal diameter of the rivets. The diameter of the die shall not exceed the diameter of the punch by more than 3/32-inch. If any holes must be enlarged to admit the rivets, they shall be reamed. Holes must be clean cut, without torn or ragged edges. Poor matching of holes may be cause for rejection.

215. **Sub-punched and Reamed Holes.**—In sub-punched and reamed work, the holes shall be punched 3/16-inch smaller and, after assembling, reamed 1/16-inch larger than the nominal diameter of the rivet. The diameter of the punch used shall be 3/16-inch smaller than the nominal diameter of the rivet and the diameter of the die not more than 3/32-inch larger than the diameter of the punch. Outside burrs shall be removed with a tool making a 1/16-inch fillet.

216. **Accuracy of Punching in Reamed Work.**—In sub-punched and reamed work, the punching shall be so accurately done that, after assembling and before reaming, a cylindrical

pin $\frac{1}{8}$-inch smaller in diameter than the nominal size of the punched hole may be entered, perpendicular to the face of the member, without drifting, in at least 75 of any group of 100 contiguous holes in the same plane. If this requirement is not fulfilled, the badly punched pieces shall be rejected. If any hole will not pass a pin 3/16-inch smaller in diameter than the nominal size of the punched hole, this shall be cause for rejection.

217. **Reaming After Assembling.**—Reaming shall be done after the pieces forming a built member are assembled and so firmly bolted together that the surfaces are in close contact. Before riveting, they shall be taken apart, if necessary, and any shavings removed. When it is necessary to take the members apart for shipping or handling, the respective pieces reamed together shall be so marked that they may be reassembled in the same position in the final setting up. No interchange of reamed parts will be permitted.

218. **Accuracy of Reaming and Drilling.**—When holes are reamed or drilled, 85 of any group of 100 contiguous holes in the same plane shall, after reaming or drilling, show no offset greater than 1/32-inch between adjacent thicknesses of metal.

219. **Reamed Holes.**—Reamed holes shall be cylindrical, perpendicular to the member, and not more than 3/32-inch larger than the nominal diameter of the rivets. Reamers preferably shall not be directed by hand. Outside burrs shall be removed with a tool making a 1/16-inch fillet.

220. **Drilled Holes.**—Drilled holes shall be 1/16-inch larger than the nominal size of the rivet. Burrs on the outside surfaces shall be removed.

221. **Assembling for Drilling.**—Connecting parts requiring drilled holes shall be assembled and securely held together while being drilled.

222. **Shop Assembling.**—The parts of riveted members shall be well pinned and firmly drawn together with bolts before riveting is commenced. The drifting done during assembling shall be only such as to bring the parts into position, and not sufficient to enlarge the holes or distort the metal. Surfaces in contact shall be painted. Bolts in field connection holes shall be left in place.

223. **Field Connections.**—Solid floor sections shall be assembled to the girders or trusses, or to suitable frames, in the shop, and the end connections made to fit. (103).

224. In reamed work, riveted trusses and skew portals shall be assembled in the shop, the parts adjusted to line and fit, and the holes for field connections drilled or reamed while so assembled. Holes for other field connections, except those in lateral, longitudinal and sway bracing, shall be drilled or reamed in the shop with the connecting parts assembled, or else drilled or reamed to a metal template.

225. In punched work, the field connections (except those in lateral, longitudinal and sway bracing) shall be reamed to metal templates.

226. **Match-marking.**—Connecting parts assembled in the shop for the purpose of reaming or drilling holes in field connections shall be match-marked, and a diagram showing such marks shall be furnished the Engineer.

227. **Rivets.**—The size of rivets called for on the plans shall be the size of the rivet before heating.

228. Rivet heads, when not countersunk or flattened, shall be of approved shape and of uniform size for the same diameter of rivet. Rivet heads shall be full, neatly made, concentric with the rivet holes, and in full contact with the surface of the member.

229. **Riveting.**—Rivets shall be heated uniformly to a light cherry red and driven while hot. Rivets, when heated and ready for driving, shall be free from slag, scale and carbon deposit. When driven, they shall completely fill the holes. Loose, burned or otherwise defective rivets shall be replaced. In removing rivets, care shall be taken not to injure the adjacent metal, and, if necessary, they shall be drilled out. Caulking or re-cupping will not be permitted.

230. Rivets shall be driven by direct-acting riveters where practicable. The riveters shall retain the pressure after the upsetting is completed.

231. When necessary to drive rivets with a pneumatic riveting hammer, a pneumatic bucker shall be used for holding up, when practicable.

232. **Field Rivets.**—Field rivets shall be furnished in excess of the nominal number required to the amount of 15 per cent plus ten rivets, for each size and length.

233. Field rivets shall be carefully selected, and shall be free from fins on the under side of the head.

234. **Turned Bolts.**—Where turned bolts are used to transmit shear, the holes shall be reamed parallel and the bolts shall make a tight fit with the threads entirely outside of the holes. A washer not less than $\frac{1}{4}$-inch thick shall be used under each nut.

235. **Planing Sheared Edges.**—Sheared edges of material more than $\frac{5}{8}$-inch in thickness and carrying calculated stress shall be planed to a depth of $\frac{1}{4}$-inch. Re-entrant cuts shall be filleted before cutting.

236. **Lacing Bars.**—The ends of lacing bars shall be neatly rounded, unless otherwise called for.

237. **Fit of Stiffeners.**—Stiffeners under the top flanges of deck girders and at all bearing points shall be milled or ground to bear against the flange angles. Other stiffeners must fit sufficiently tight against the flange angles to exclude water after being painted. Fillers and splice plates shall fit within $\frac{1}{4}$-inch at each end.

238. **Web Plates.**—Web plates of girders which have no cover plates may be $\frac{1}{8}$-inch above or below the backs of the top flange angles. Web plates of girders which have cover plates may be $\frac{1}{2}$-inch less in width than the distance back to back of flange angles.

239. When web plates are spliced, not more than $\frac{3}{8}$-inch clearance between ends of plates will be allowed.

240. **Facing Floorbeams, Stringers and Girders.**—Floorbeams, stringers and girders having end connection angles shall be made of exact length after the connection angles are riveted. If facing is necessary, the thickness of the angles shall not be reduced more than $\frac{1}{8}$-inch at any point.

241. **Finished Members.**—Finished members shall be true to line and free from twists, bends and open joints.

242. **Abutting Joints.**—Abutting joints in compression members, and girder flanges, and, where so specified on the drawings, in tension members, shall be faced and brought to an even bearing. Where joints are not faced, the opening shall not exceed $\frac{1}{4}$-inch.

243. **Eye-bars.**—Eye-bars shall be straight, true to size, and free from twists, folds in the neck or head, and other defects. The heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of the heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the Engineer. The thickness of the head and neck shall not overrun more than 1/16-inch for bars 8 inches or less in width, $\frac{1}{8}$-inch for bars more than 8 inches and not more than 12 inches in width, and 3/16-inch for bars more than 12 inches wide.

244. Eye-bars which are to be placed side by side in the structure shall be bored so accurately that, upon being placed together, the pins will pass through the holes at both ends at the same time without driving. Eye-bars shall have both ends bored at the same time.

245. **Annealing.**—Eye-bars shall be annealed by heating uniformly to the proper temperature followed by slow and uniform cooling. Proper instruments shall be provided for determining at all times the temperature of the bars.

246. Other steel which has been partially heated shall be properly annealed except where used in minor parts.

247. **Boring Pin Holes.**—Pin holes shall be bored true to gage, smooth, straight, at right angles with the axis of the member and parallel with each other, unless otherwise required. The variation from the specified distance from outside to outside of pin holes in tension members, or from inside to inside of pin holes in compression members, shall not exceed 1/32-inch. In built-up members the boring shall be done after the member is riveted.

248. **Boring Pins.**—Pins larger than 9 inches in diameter shall have a hole bored longitudinally through the center of each not less than 2 inches in diameter.

249. **Pin Clearances.**—The difference in diameter between the pin and the pin hole shall be 1/50-inch for pins up to 5 inches in diameter, and 1/32-inch for larger pins.

250. **Pins and Rollers.**—Pins and rollers shall be accurately turned to gage and shall be straight, smooth and free from flaws.

251. **Screw Threads.**—Screw threads shall make close fits in the nuts and shall be U. S. Standard, except that for pin ends of diameters greater than $1\frac{3}{8}$ inches, they shall be made with six threads to an inch.

252. **Welds.**—Welds in steel will not be allowed, except to remedy minor defects.

253. **Forging Pins.**—Pins larger than 7 inches in diameter shall be forged and annealed.

254. **Bearing Surfaces Planed.**—The top and the bottom surfaces of base and cap plates of columns and pedestals, except those in contact with masonry, shall be planed, or hot-straightened, and parts of members in contact with them shall be faced to fit. Connection angles for base plates and cap plates shall be riveted to compression members before the members are faced.

255. Sole plates of plate girders shall have full contact with the girder flanges. Sole plates and masonry plates shall be planed or hot-straightened. Cast pedestals shall be planed on the surfaces in contact with steel and shall have the bottom surfaces resting on masonry rough finished.

256. **Pilot Nuts.**—Two pilot nuts and two driving nuts shall be furnished for each size of pin, unless otherwise specified.

XIII. WEIGHING AND SHIPPING

257. **Weight Paid for.**—The payment for pound price contracts shall be based on the scale weight of the metal in the fabricated structure, including field rivets shipped. The weight of the field paint and cement, if furnished, boxes and barrels used for packing, and material used for staying or supporting members on cars, shall be excluded.

258. **Variation in Weight.**—If the weight of any member is more than $2\frac{1}{2}$ per cent less than the computed weight, it may be cause for rejection.

259. The greatest allowable variation of the total scale weight of any structure from the weights computed from the approved shop drawings shall be $1\frac{1}{2}$ per cent. Any weight in excess of $1\frac{1}{2}$ per cent above the computed weight shall not be paid for by the Company.

260. **Computed Weight.**—The weight of steel shall be assumed at 0.2833 lb. per cubic inch.

261. The weights of rolled shapes, and of plates, up to and including 36 inches in width, shall be computed on the basis of their nominal weights and dimensions, as shown on the approved shop drawings, deducting for copes, cuts and open holes.

262. The weights of plates wider than 36 inches shall be computed on the basis of their dimensions, as shown on the approved shop drawings, deducting for cuts and open holes. To this shall be added one-half of the allowed percentages of overrun in weight given in Article 180.

263. The weight of heads of shop driven rivets shall be included in the computed weight.

264. The weights of castings shall be computed from the dimensions shown on the approved shop drawings, with an addition of 10 per cent for fillets and overrun.

265. **Weighing of Members.**—Finished work shall be weighed in the presence of the Inspector, if practicable. The Contractor shall furnish satisfactory scales and do the handling of the material for weighing.

266. **Marking and Shipping.**—Members weighing more than 5 tons shall have the weight marked thereon. Bolts and rivets of one length and diameter, and loose nuts or washers of each size, shall be packed separately. Pins, other small parts, and small packages of bolts, rivets, washers and nuts shall be shipped in boxes, crates, kegs or barrels, but the gross weight of any package shall not exceed 300 pounds. A list and description of the contained material shall be plainly marked on the outside of each package, box or crate.

267. Long girders shall be so loaded and marked that they may arrive at the bridge site in position for erection without turning.

268. Anchor bolts, washers and other anchorage or grillage materials shall be shipped in time for them to be built into the masonry.

XIV. SHOP PAINTING

269. **Shop Cleaning and Painting.**—Unless otherwise specified, steel work, after it has been accepted by the Inspector and before leaving the shop, shall be thoroughly cleaned and given one coat of approved paint, applied in a workmanlike manner and well worked into joints and open spaces. Cleaning shall be done with steel brushes, hammers, scrapers and chisels, or by other equally effective means. Oil, paraffin and grease shall be removed by wiping with benzine or gasoline. Loose dirt shall be brushed off with a dry bristle brush before the paint is applied.

270. **Surfaces in Contact.**—Surfaces coming in contact shall be cleaned and given one coat of paint on each surface before assembling.

271. **Erection Marks.**—Erection marks shall be painted on painted surfaces.

272. **Painting in Damp or Freezing Weather.**—Painting shall not be done in damp or freezing weather except under cover, and the steel must be free from moisture or frost when the paint is applied. Material painted under cover in damp or freezing weather shall be kept under cover until the paint is dry.

273. **Mixing of Paint.**—Paint shall be thoroughly mixed before applying, and the pigments shall be kept in suspension.

274. **Machine Finished Surfaces.**—Machine surfaces of steel (except abutting joints and base plates) shall be coated with white lead and tallow, applied hot as soon as the surfaces are finished and accepted by the Inspector.

XV. MILL AND SHOP INSPECTION

275. **Facilities for Inspection.**—Facilities for inspection of material and workmanship in the mill and shop shall be furnished by the Contractor to the Inspectors, and the Inspectors shall be allowed free access to the necessary parts of the premises.

276. **Mill Orders and Shipping Statements.**—The Contractor shall furnish the Engineer with as many copies of material orders and shipping statements as the Engineer may direct. The weights of the individual members shall be shown.

277. **Notice of Rolling.**—The Contractor shall give ample notice to the Engineer of the beginning of rolling at the mill, and of work at the shop, so that inspection may be provided. No material shall be rolled nor work done before the Engineer has been notified where the orders have been placed.

278. **Cost of Testing.**—The Contractor shall furnish, without charge, test specimens, as specified herein, and all labor, testing machines and tools necessary to make the specimen and full size tests.

279. **Inspector's Authority.**—The Inspector shall have the power to reject materials or workmanship which do not come up to the requirements of these specifications; but in cases of dispute, the contractor may appeal to the Engineer, whose decision shall be final.

280. **Rejections.**—The acceptance of any material or finished members by the Inspector shall not be a bar to their subsequent rejection, if found defective.

281. Rejected material and workmanship shall be replaced promptly or made good by the Contractor.

XVI. FULL-SIZE TESTS

282. **Full-Size Tests of Eye-Bars.**—The number and size of the bars to be tested shall be stipulated by the Engineer before the mill order is placed. The number shall not exceed 5 per cent of the whole number of bars ordered, with a minimum of two bars on small orders.

283. The test bars shall be of the same section as the bars to be used in the structure and of the same length if within the capacity of the testing machine. They shall be selected by the Inspector from the finished bars preferably after annealing. Test bars representing bars too long for the testing machine shall be selected from the full length bar material after the heads on one end have been formed and shall have the second head formed upon them after being cut to the greatest length which can be tested.

284. Full-size tests of eye-bars shall show a yield point of not less than 29,000 pounds per square inch, an ultimate strength of not less than 54,000 pounds per square inch, and an elongation of not less than 10 per cent in a length of 20 feet measured in the body of the bar. The fracture shall show a silky or finely granular structure throughout.

285. If a bar fails to meet the requirements of Article 284, two additional bars of the same size and from the same mill heat shall be tested. If the failure of the first test bar is on account of the character of the fracture only, the bars represented by the test may be reannealed before the additional bars are tested.

286. If two of the three bars tested fail, the bars of that size and mill heat shall be rejected.

287. A failure in the head of a bar shall not be cause for rejection if the other requirements are fulfilled.

288. A record of the annealing charges shall be furnished the Engineer showing the bars included in each charge and the treatment they receive.

289. Bars thus tested which meet the requirements of the specifications shall be paid for by the Company at the same unit prices as the structures. Bars which fail to meet the requirements of the specifications, and all bars rejected as a result of tests, shall be at the Contractor's expense.

REFERENCES.—For the calculation of the stresses in railway bridges and for additional details and the details of design, the following books may be consulted: Merriman & Jacoby's "Roofs and Bridges," Part I, Stresses; Part II, Graphic Statics; Part III, Bridge Design; Part IV, Higher Structures; Johnson, Bryan and Turneaure's "Framed Structures," Part I, Stresses, Part II, Statically Indeterminate Structures and Secondary Stresses; Part III, Design; Marburg's "Framed Structures," Part I, Stresses; Spofford's "Theory of Structures," stresses in structures; DuBois's "Framed Structures"; Burr and Falk's "Design and Construction of Metallic Bridges"; Skinner's "Details of Bridge Design," Parts I, II, III; Moore's "Design of Plate Girders"; Kunz's "Design of Steel Railway Bridges; Ketchum's "The Design of Highway Bridges of Steel, Timber and Concrete," stresses, details and design.

CHAPTER V.

RETAINING WALLS.

Introduction.—A retaining wall is a structure which sustains the lateral pressure of earth or some other granular mass which possesses some frictional stability. The pressure of the material supported will depend upon the material, the manner of depositing in place, and upon the amount of moisture, and will vary from zero to the full hydraulic pressure. If dry clay is loosely deposited behind the wall it will exert full pressure, due to this condition. In time the earth may become consolidated and cohesion and moisture make a solid clay, which may cause the bank to shrink away from the wall and there will be no pressure exerted. On the other hand all cohesion may be destroyed by the vibration of moving loads or by saturation, and the maximum theoretical pressures may occur. The pressures due to a dry granular mass, a semi-fluid, without cohesion, of indefinite extent, the particles held in place by friction on each other, will be considered. The effect of cohesion and of limiting the extent of the mass is considered in the author's "The Design of Walls, Bins and Grain Elevators."

Nomenclature.—The following nomenclature will be used:

ϕ = the angle of repose of the filling.
ϕ' = the angle of friction of the filling on the back of the wall.
θ = the angle between the back of the wall and a horizontal line passing through the heel of the wall and extending from the back into the fill.
δ = angle of surcharge, the angle between the surface of the filling and the horizontal; δ is positive when measured above and negative when measured below the horizontal.
z = the angle which the resultant earth-pressure makes with a normal to the back of the wall.
λ = the angle between the resultant thrust, P, and a horizontal line.
h = the vertical height of the wall in feet.
d = the width of the base of the wall in feet.
b = the distance from the center of the base to the point where the resultant pressure, E, cuts the base.
P = the resultant earth-pressure per foot of length of wall.
E = the resultant of the earth-pressure and the weight of the wall.
w = the weight of the filling per cubic foot.
W = the total weight of the wall per foot of length of wall.
p_1 = the pressure on the foundation due to direct pressure.
p_2 = the pressure on the foundation due to bending moments.
p = the resultant pressure on the foundation due to direct and bending forces.
y = the depth of foundation below the earth surface.

Calculation of the Pressure on Retaining Walls.—To fully determine the pressure of the filling on a retaining wall it is necessary that the resultant of the pressure be known (*a*) in amount, (*b*) in line of action, and (*c*) in point of application. Many theories have been proposed for finding the pressure, each differing somewhat as to the assumptions and results. All theories for the design of retaining walls that have any theoretical basis come in two classes: (1) the Theory of Conjugate Pressures, due to Rankine, and commonly known as Rankine's Theory, and (2) the Theory of the Maximum Wedge, probably first proposed by Coulomb, and commonly known as Coulomb's Theory. Rankine's Theory determines the thrust in amount, in line of action, and in point of application. In Coulomb's Theory, with the exception of Weyrauch's solution, the line of action and point of application must be assumed, thus leading to numerous solutions of

more or less merit. All solutions based on the theory of the wedge assume that the resultant thrust is applied at one-third the height for a wall with a level or inclined surcharge, as is given by Rankine; but the resultant is assumed as making angles with a normal to the back of the wall varying from zero to the angle of repose of the filling. In Rankine's solution the resultant pressure is parallel to the plane of the surcharge for a vertical wall with a level or positive surcharge.

(1) **RANKINE'S THEORY.**—In this theory the filling is assumed to consist of an incompressible, homogeneous, granular mass, without cohesion, the particles are held in position by friction on each other; the mass being of indefinite extent, having a plane top surface, resting on a homogeneous foundation, and being subjected to its own weight. The principal and conjugate stresses in the mass are calculated, thus leading to the ellipse of stress. In the analysis it is proved (*a*) that the maximum angle between the pressure on any plane and the normal to the plane is equal to the angle of internal friction, and (*b*) that there is no active upward component of stress in a granular mass. Both of these laws have been verified by experiments on semi-fluids. Rankine deduced algebraic formulas for calculating the resultant pressure on a vertical wall with a horizontal surcharge, and on a vertical wall with a surcharge equal to δ, an angle equal to or less than the angle of repose. The general case is best solved by constructing the ellipse of stress by graphics, or Weyrauch's algebraic solution may be used. The author has extended Rankine's solution in "The Design of Walls, Bins and Grain Elevators," so that it is perfectly general.

Rankine's Formulas.—With a vertical wall and a horizontal surcharge, Fig. 1, the total resultant pressure is

$$P = \tfrac{1}{2}w\cdot h^2 \frac{1 - \sin\phi}{1 + \sin\phi} \tag{1}$$

where w is the weight of the filling in lb. per cu. ft., h is the depth of the wall in feet, ϕ is the angle of repose of the filling, and P is the resultant pressure on the wall in pounds. The resultant pressure, P, will be horizontal.

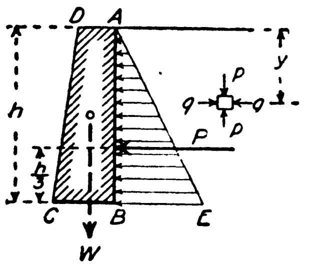


FIG. 1.

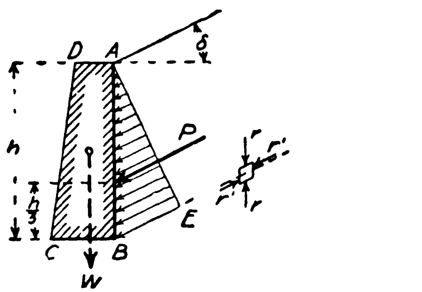


FIG. 2.

For a vertical wall with surcharge at an angle δ, Fig. 2, the pressure is given by the formula

$$P = \tfrac{1}{2}w\cdot h^2\cdot\cos\delta \frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}} \tag{2}$$

Where δ is equal to ϕ, formula (2) becomes

$$P = \tfrac{1}{2}w\cdot h^2 \cos\phi \tag{3}$$

The resultant pressure, P, is parallel to the inclined top surface for a vertical wall with a level or a positive surcharge (many authors have incorrectly assumed that the resultant pressure is always parallel to the top surface of the surcharged filling).

Inclined Retaining Wall.—The pressure on an inclined retaining wall may be calculated by means of the ellipse of stress—see the author's "The Design of Walls, Bins and Grain Elevators."

The pressure on an inclined retaining wall may also be calculated by means of the graphic solution shown in Fig. 3 if the direction of the thrust be known. From Rankine's theory we know that the resultant pressure on a vertical retaining wall is always parallel to the top surface where the surcharge is level or is inclined upwards away from the wall. The pressure on a retaining wall inclined away from the filling may then be calculated as follows:

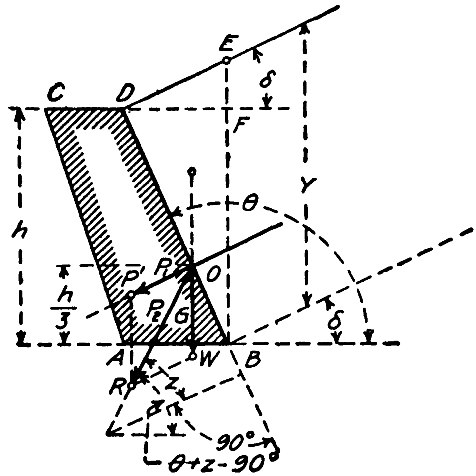


FIG. 3. PRESSURE ON AN INCLINED RETAINING WALL.

In Fig. 3 the retaining wall $ACDB$ sustains the pressure of a filling having an angle of repose ϕ, and sloping up away from the top of the wall at an angle δ. Calculate P' the pressure on the plane E–B by means of formula (2). P' acts at a point $\frac{1}{3}EB$ above B and is parallel to the top surface DE. Let the weight of the triangle of filling DBE be G, which acts through the center of gravity of the triangle and intersects P' at point O. Then P_2, the resultant of P' and G, will be the resultant pressure at O, and makes an angle z with a normal to the back of the wall, and an angle, $\lambda = \theta + z - 90°$ with the horizontal.

(2) **COULOMB'S THEORY.**—In this theory it is assumed that there is a wedge having the wall as one side and a plane called the plane of rupture as the other side, which exerts a maximum thrust on the wall. The plane of rupture lies between the angle of repose of the filling and the back of the wall. It may coincide with the plane of repose. For a wall without surcharge (horizontal surface back of the wall) and a vertical wall the plane of rupture bisects the angle between the plane of repose and the back of the wall. This theory does not determine the direction of the thrust, and leads to many other theories having assumed directions for the resultant pressure.

Algebraic Method.—In Fig. 4, the wall with a height h, slopes toward the earth, being inclined to the horizontal at an angle θ, and the earth has a surcharge with slope δ, which is not greater than ϕ, the angle of repose. It is required to find the pressure P against the retaining wall, it being assumed that the resultant pressure makes an angle z with the back of the wall.

It is assumed that the triangular prism of earth above some plane, the trace of which is the line AE, will produce the maximum pressure on the wall and on the earth below the plane, and that in turn the prism will be supported by the reactions of the wall and the earth. Let OW represent the weight of the prism ABE, the length of the prism being assumed equal to unity, let OP be the reaction of the wall, and OR be the reaction of the earth below.

Now the forces OW, OP, and OR will be concurrent and will be in equilibrium; OP and OR will therefore be components of OW. When the prism ABE is just on the point of moving OP

will make an angle with a normal to the back of the wall equal to z (different authorities assume values of z from zero to ϕ', the angle of friction of earth on masonry, or ϕ, the angle of repose of earth); while OR will make an angle with the normal to the plane of rupture AE equal to ϕ. Let P represent the pressure OP against the wall, W represent the weight of the prism of earth, and w the weight per cu. ft.

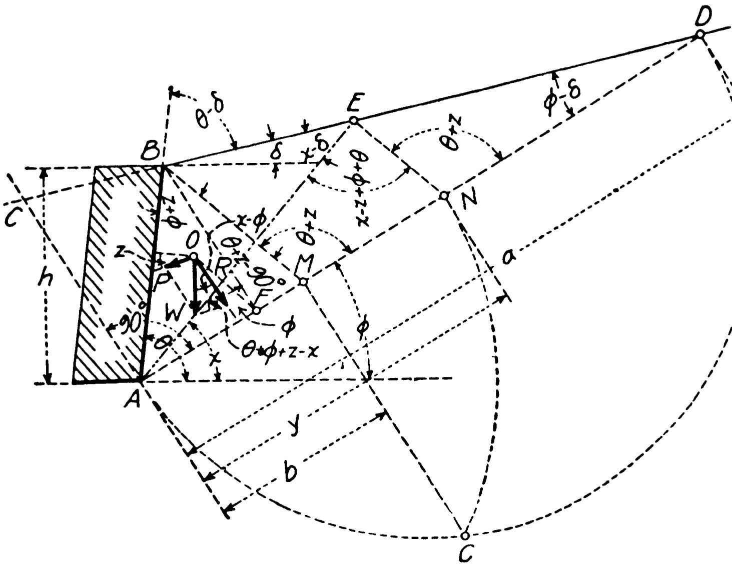


FIG. 4.

In the triangle OWR angle $WOR = x - \phi$, and angle $ORW = \theta + \phi + z - x$. Through E draw EN, making the angle $AEN = \theta + \phi + z - x$ with AE. Then the triangle AEN is similar to triangle ORW, and

$$\frac{P}{W} = \frac{EN}{AN}, \quad \text{and} \quad P = W\frac{EN}{AN}$$

But W equals $w \cdot$ *area triangle* $ABE = \frac{1}{2}w \cdot AB \cdot BE \cdot \sin(\theta - \delta)$, and

$$P = \tfrac{1}{2}w \cdot \sin(\theta - \delta)\frac{AB \cdot BE \cdot EN}{AN} \tag{4}$$

Now P varies with the angle x, and will have a maximum value for some value of x, which may be found by differentiating (4) and placing the result equal to zero.

Differentiating and substituting in (4) and reducing we have

$$P = \tfrac{1}{2}w \cdot h^2 \frac{\sin^2(\theta - \phi)}{\sin^2\theta \cdot \sin(\theta + z)\left(1 + \sqrt{\dfrac{\sin(z + \phi) \cdot \sin(\phi - \delta)}{\sin(\theta + z) \cdot \sin(\theta - \delta)}}\right)^2} \tag{5}$$

$$= \tfrac{1}{2}w \cdot h^2 \cdot K \tag{6}$$

which is the general formula for the pressure on a retaining wall.

Now if z in (5) is made equal to ϕ', the angle of repose of earth on the wall,

$$P = \tfrac{1}{2}w \cdot h^2 \frac{\sin^2(\theta - \phi)}{\sin^2\theta \cdot \sin(\theta + \phi')\left(1 + \sqrt{\dfrac{\sin(\phi + \phi') \cdot \sin(\phi - \delta)}{\sin(\theta + \phi') \cdot \sin(\theta - \delta)}}\right)^2} \tag{7}$$

which is Cain's formula (20) in another form.

If z in (5) is made equal to δ, and θ made equal to 90°,

$$P = \tfrac{1}{2}w \cdot h^2 \frac{\cos^2 \phi}{\cos \delta \left(1 + \sqrt{\frac{\sin (\phi + \delta) \cdot \sin (\phi - \delta)}{\cos^2 \delta}} \right)^2} \tag{8}$$

which is Rankine's formula (2) in another form.

If z in (5) is made equal to zero,

$$P = \tfrac{1}{2}w \cdot h^2 \frac{\sin^2 (\theta - \phi)}{\sin^3 \theta \left(1 + \sqrt{\frac{\sin \phi \cdot \sin (\phi - \delta)}{\sin \theta \cdot \sin (\theta - \delta)}} \right)^2} \tag{9}$$

which gives the normal pressure on a wall.

If θ in (9) = 90°,

$$P = \tfrac{1}{2}w \cdot h^2 \frac{\cos^2 \phi}{\left(1 + \sqrt{\frac{\sin \phi \cdot \sin (\phi - \delta)}{\cos \delta}} \right)^2} \tag{10}$$

If δ in (10) = 0°,

$$P = \tfrac{1}{2}w \cdot h^2 \frac{\cos^2 \phi}{(1 + \sin \phi)^2},$$

$$= \tfrac{1}{2}w \cdot h^2 \tan^2 (45° - \tfrac{1}{2}\phi) \tag{11}$$

$$= \tfrac{1}{2}w \cdot h^2 \frac{1 - \sin \phi}{1 + \sin \phi} \tag{12}$$

which is Rankine's formula (1) for a vertical wall without surcharge.

Graphic Method.—If the angle z, the angle between the line of pressure and a normal to the wall, is known, the resultant pressure on a wall may be calculated by a graphic method, Fig. 5, based on the "theory of a wedge of maximum thrust." The graphic method will be described—the proof of the method is given in "The Design of Walls, Bins and Grain Elevators."

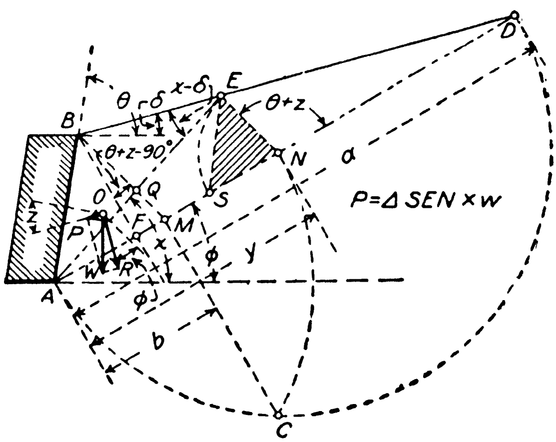


FIG. 5.

In Fig. 5 the retaining wall AB sustains the pressure of the filling with a surcharge δ and an angle of repose ϕ. It is required to calculate the resultant pressure P.

The graphic solution is as follows: Through B in Fig. 5 draw BM making an angle with BF, the normal to AD, equal to $\lambda = \theta + z - 90°$, the angle that P makes with the horizontal. With

diameter AD describe arc ACD. Draw MC normal to AD and with A as a center and a radius AC describe arc CN. Then $AN = y$, $AM = b$ and $y = \sqrt{a \cdot b}$. Draw EN parallel to BM. With N as a center and radius EN, describe arc ES. Then AE is the trace of the plane of rupture, and P = area $SEN \cdot w$.

Cain's Formulas.*—Professor William Cain assumes that the angle z is equal to ϕ', the angle of friction of the filling on the back of the wall. By substituting in (5) we have for a

Vertical Wall With Level Surface, $\delta = 0$.

$$P = \tfrac{1}{2} w \cdot h^2 \left(\frac{\cos \phi}{n + 1} \right)^2 \frac{1}{\cos \phi'} \tag{13}$$

where

$$n = \sqrt{\frac{\sin (\phi + \phi') \cdot \sin \phi}{\cos \phi'}}$$

If $\phi = \phi'$, then $n = \sqrt{2} \sin \phi$, and

$$P = \tfrac{1}{2} w \cdot h^2 \frac{\cos \phi}{(1 + \sin \phi \sqrt{2})^2} \tag{14}$$

If $\phi' = 0$, then

$$P = \tfrac{1}{2} w \cdot h^2 \cdot \tan^2 \left(45^\circ - \frac{\phi}{2} \right) \tag{15}$$

Vertical Wall With Surcharge $= \delta$.

$$P = \tfrac{1}{2} w \cdot h^2 \left(\frac{\cos \phi}{n + 1} \right)^2 \frac{1}{\cos \phi'} \tag{16}$$

where

$$n = \sqrt{\frac{\sin (\phi + \phi') \cdot \sin (\phi - \delta)}{\cos \phi' \cdot \cos \delta}}$$

If $\delta = \phi$,

$$P = \tfrac{1}{2} w \cdot h^2 \frac{\cos^2 \phi}{\cos \phi'} \tag{17}$$

If $\phi' = 0$, and $\delta = \phi$,

$$P = \tfrac{1}{2} w \cdot h^2 \cdot \cos^2 \phi \tag{18}$$

Inclined Wall With Horizontal Surface.

$$P = \tfrac{1}{2} w \cdot h^2 \left(\frac{\sin (\theta - \phi)}{(n + 1) \sin \theta} \right)^2 \frac{1}{\sin (\phi' + \theta)} \tag{19}$$

where

$$n = \sqrt{\frac{\sin (\phi + \phi') \cdot \sin \phi}{\sin (\phi' + \theta) \cdot \sin \theta}}$$

Inclined Wall With Surcharge $= \delta$.

$$P = \tfrac{1}{2} w \cdot h^2 \left(\frac{\sin (\theta - \phi)}{(n + 1) \cdot \sin \theta} \right)^2 \frac{1}{\sin (\phi' + \theta)} \tag{20}$$

where

$$n = \sqrt{\frac{\sin (\phi + \phi') \cdot \sin (\phi - \delta)}{\sin (\phi' + \theta) \cdot \sin (\theta - \delta)}}$$

Wall With Loaded Filling.—In Fig. 6, the filling is loaded with a uniformly distributed load. Calculate h_1 by dividing the loading per sq. ft. by w. Let $h + h_1 = H$. Then the resultant pressure for a wall with height H, will be

$$P_2 = \tfrac{1}{2} w \cdot H^2 \cdot K \tag{21}$$

and the resultant pressure for a wall with height h_1, will be

$$P_1 = \tfrac{1}{2} w \cdot h_1^2 \cdot K \tag{22}$$

* Professor Rebhann makes the same assumptions and uses the graphic method of Fig. 5.

The pressure on the wall AD will be

$$P = P_2 - P_1 = \tfrac{1}{2}w(H^2 - h_1^2)K \tag{23}$$

and the point of application is through the center of gravity of $ADGE$, which makes

$$y_1 = \tfrac{1}{3}\,\frac{H^2 + H\cdot h_1 - 2h_1^2}{H + h_1} \tag{24}$$

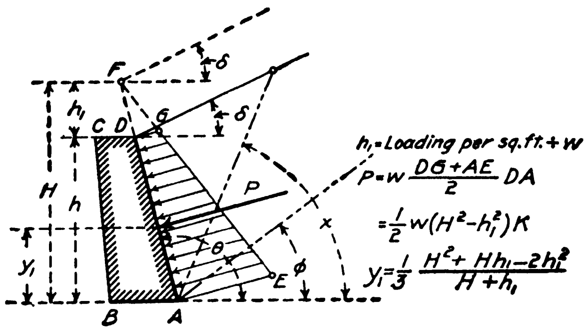


FIG. 6.

Walls With Negative Surcharge.—For the calculation of the pressures on retaining walls with negative surcharge, δ negative, see the author's "The Design of Walls, Bins and Grain Elevators," second edition.

STABILITY OF RETAINING WALLS.—A retaining wall must be stable (1) against overturning, (2) against sliding, and (3) against crushing the masonry or the foundation.

The factor of safety of a retaining wall is the ratio of the weight of a filling having the same angle of internal friction that will just cause failure to the actual weight of the filling. For a factor of safety of 2 the wall would just be on the point of failure with a filling weighing twice that for which the wall is built.

1. **Overturning.**—In Fig. 7, let P, represented by OP', be the resultant pressure of the earth, and W, represented by OW, be the weight of the wall acting through its center of gravity. Then E, represented by OR, will be the resultant pressure tending to overturn the wall.

Draw OS through the point A. For this condition the wall will be just on the point of overturning, and the factor of safety against overturning will be unity. The factor of safety for $E = OR$ will be

$$f_0 = SW/RW \tag{25}$$

2. **Sliding.**—In Fig. 7 construct the angle $H1G$ equal to ϕ', the angle of friction of the masonry on the foundation. Now if E passes through 1, and takes the direction OQ, the wall will be on the point of sliding, and the factor of safety against sliding, f_s, will be unity. For $E = OR$, the factor of safety against sliding will be

$$f_s = QM'/RM \tag{26}$$

Retaining walls seldom fail by sliding.

The factor of safety against sliding is sometimes given as

$$f_s = \frac{F}{H}\tan\phi'. \tag{27}$$

where H is the horizontal component of P. Equations (26) and (27) give the same values only where the resultant P is horizontal.

3. **Crushing.**—In Fig. 7 the load on the foundation will be due to a vertical force F, which produces a uniform stress, $p_1 = F/d$, over the area of the base, and a bending moment $= F\cdot b$, which produces compression, p_2, on the front and tension, p_2, on the back of the foundation.

The sum of the tensile stresses due to bending must equal the sum of the compressive stresses, $= \frac{1}{4}p_2d$. These stresses act as a couple through the centers of gravity of the stress triangles on each side, and the resisting moment is

$$M' = \tfrac{1}{4}p_2 \cdot d \cdot \tfrac{2}{3}d = \tfrac{1}{6}p_2 \cdot d^2 \tag{28}$$

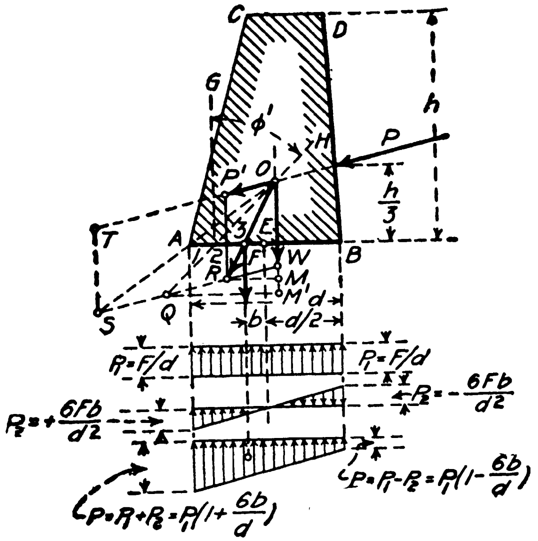


FIG. 7.

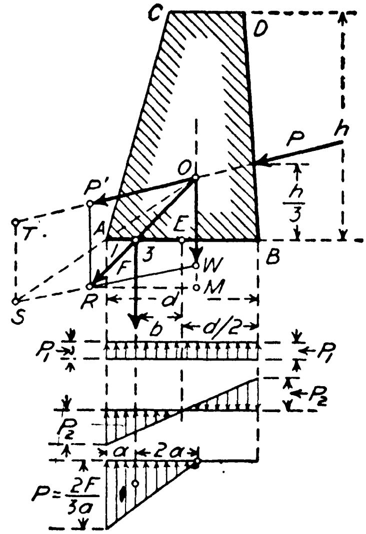


FIG. 8.

But the resisting movement equals the overturning moment, and

$$\tfrac{1}{6}p_2 \cdot d^2 = F \cdot b,$$

and

$$p_2 = \pm \frac{6F \cdot b}{d^2} \tag{29}$$

The total stress on the foundation then is

$$p = p_1 \pm p_2 = p_1(1 \pm 6b/d) \tag{30}$$

Now if $b = \frac{1}{6}d$, we will have

$$p = 2p_1, \quad \text{or} \quad 0.$$

In order therefore that there be no tension, or that the compression never exceed twice the average stress, the resultant should never strike outside the middle third of the base.

If the resultant strikes outside of the middle third of a wall in which the masonry can take no tension, the load will all be taken by compression and can be calculated as follows:

In Fig. 8 the resultant F will pass through the center of gravity of the stress diagram, and will equal the area of the diagram.

$$F = \tfrac{3}{2}p \cdot a$$

and

$$p = \frac{2F}{3a} \tag{31}$$

which gives a larger value of p than would be given if the masonry could take tension.

General Principles of Design.—The overturning moment of a masonry retaining wall of gravity section depends upon the weight of the filling, the angle of internal friction of the filling, the surcharge, and the height and shape of the wall. The resisting moment depends upon the

weight of the masonry, the width of the foundation, and the cross-section of the wall. The most economical section for a masonry retaining wall is obtained when the back slopes toward the filling. In cold localities, however, this form of section may be displaced by heaving due to the action of frost, and it is usual to build retaining walls with a slight batter forwards. The front of the wall is usually built with a batter of from ½ in. to 1 in. in 12 in. In order to keep the center of gravity of the wall back of the center of the base it is necessary to increase the width of the wall at the base by adding a projection to the front side. Where the wall is built on the line of a right of way it is sometimes necessary to increase the width of the base by putting the projection on the rear side, making an L-shaped wall. The weight of the filling upon the base and back of the wall adds to the stability of the wall. Where the wall is built to support an embankment expensive to excavate, it is often economical to make the wall L-shaped, with all the projection on the front side.

In calculating the thrust on retaining walls great care must be exercised in selecting the proper values of w and ϕ, and the conditions of surcharge. It will be seen from the preceding discussion that the value of the thrust increases very rapidly as ϕ decreases, and as the surcharge increases. Where the wall is to sustain an embankment carrying a railroad track, buildings, or other loads, a proper allowance must be made for the surcharge.

The filling back of the wall should be deposited and tamped in approximately horizontal layers, or with layers sloping back from the wall; and a layer of sand, gravel or other porous material should be deposited between the filling and the wall, to drain the filling downwards. To insure drainage of the filling, drains should be provided back of the wall and on top of the footing, and "weep-holes" should be provided near the bottom of the wall at frequent intervals to allow the water to pass through the wall. With walls from 15 to 25 ft. high, it is usual to use "weepers" 4 in. in diameter placed from 15 to 20 ft. apart. The "weepers" should be connected with a longitudinal drain in front of the wall. The filling in front of the wall should also be carefully drained.

The permissible point at which the resultant thrust may strike the base of the foundation will depend upon the material upon which the retaining wall rests. When the foundation is solid rock or the wall is on piles driven to a good refusal, the resultant thrust may strike slightly outside the middle third with little danger to the stability of the wall. When the retaining wall, however, rests upon compressible material the resultant thrust should strike at or inside the center of the base. Where the resultant thrust strikes outside of the center of the base, any settlement of the wall will cause the top to tip forward, causing unsightly cracks and local failure in many cases, and total failure where the settlement is excessive. Where extended footings are used it may be necessary to use some reinforcing steel to prevent a crack in the footing in line with the face of the wall.

Plain masonry walls should be built in sections, the length depending upon the height of the wall, the foundation and other conditions.

Under usual conditions the length of the sections should not exceed 40 ft., 30 ft. sections being preferable, and in no case should the length of the section exceed about three times the height. Separate sections should be held in line and in elevation, either by grooves in the masonry or by means of short bars placed at intervals in the cross-section of the wall, fastened rigidly in one section and sliding freely in the other. The back of the expansion joints should be waterproofed with 3 or 4 layers of burlap and coal tar pitch. The burlap should be about 30 in. wide, and the pitch and the burlap should be applied as on tar and gravel roofs. The joints between the sections of a retaining wall on the front side should be from ⅛ to ¼ of an in. in width, and should be formed by a V-shaped groove made of sheet steel and fastened to the forms while the concrete is being placed. Where there is danger of the water in the filling percolating through the wall or in an alkali country, the surface of the back of the wall should be coated with a waterproof coating. The most satisfactory waterproof coating known to the author is a coal tar paint made by mixing refined coal tar, Portland cement and kerosene in the proportions of 16 parts refined coal tar, 4 parts of Portland cement and 3 parts of kerosene oil. The Portland

cement and kerosene should be mixed thoroughly and the coal tar then added. In cold weather the coal tar may be heated and additional kerosene added to take account of the evaporation. This paint not only covers the surface but combines with it, so that two or three coats are sometimes required. While the surface of the concrete should be dry, coal tar paint will adhere to moist or wet concrete. In building retaining walls in sections, the end of the finished section should be coated with coal tar paint to prevent the adhesion to the next section.

For methods of waterproofing masonry, see methods of waterproofing bridge floors in Chapter IV.

DESIGN OF RETAINING WALLS.—The design of masonry retaining walls will be illustrated by the design of the retaining walls for West Alameda Avenue Subway, taken from the author's "The Design of Walls, Bins and Grain Elevators," second edition.

Design of Retaining Walls for West Alameda Avenue Subway, Denver, Colorado.—The height of the walls varied from 8 ft. to 29 ft. 3 in., while the foundation soil varied from a compact gravel to a mushy clay. The design of the maximum section, which rests on a compact gravel, will be given. The concrete was mixed in the proportion of 1 part Portland cement, 3 parts sand and 5 parts screened gravel. Crocker and Ketchum, Denver, Colo., were the consulting engineers. The wall is shown in Fig. 9 and in Fig. 10.

The following assumptions were made: Weight of concrete, 150 lb. per cu. ft.; weight of filling, w = 100 lb. per cu. ft.; angle of repose of filling, $1\frac{1}{2} : 1$ ($\phi = 33° 40'$); surcharge, 600 lb. per sq. ft., equivalent to 6 ft. of filling; maximum load on foundation, 6,000 lb. per sq. ft.

Solution.—After several trials the following dimensions were taken: Width of coping 2 ft. 6 in., thickness of coping 1 ft. 6 in., batter of face of wall $\frac{1}{2}$ in. in 12 in , batter of back of wall $3\frac{1}{2}$ in. in 12 in., width of base 15 ft. $2\frac{5}{8}$ in. (ratio of base to height = 0.52), front projection of base 4 ft., other dimensions as shown in Fig. 9. The calculations were made for a section of the wall one foot in length.

The property back of the wall will probably be used for the storage of coal, etc., and it was assumed that the surcharge came even with the back edge of the footing of the wall. The resultant pressure of the filling on the plane A–2 was calculated by the graphic method of Fig. 5 and Fig. 6, and was found to be $P' = 17{,}290$ lb. The weight of the filling in the wedge back of the wall is $W' = 16{,}435$ lb., acting through the center of gravity of the filling. The resultant of P' and W' is $P = 23{,}850$ lb. = the resultant pressure of the filling on the back of the wall. The weight of the masonry is $W = 33{,}144$ lb., acting through the center of gravity of the wall, and the resultant of P and W is $E = 52{,}510$ lb. = the resultant pressure of the wall and the filling upon the foundation. The vertical component of E is $F = 49{,}580$ lb., and cuts the foundation, $b = 2.1$ ft. from the middle.

1. *Stability Against Overturning.*—The line OD in this case is nearly parallel to the line QW which brings the point S in Fig. 9 at a great distance from the point W. The factor of safety against overturning was calculated on the original drawing and found to be $f_0 > 25$.

2. *Stability Against Sliding.*—The coefficient of friction of the masonry on the footing will be assumed to be $\tan \phi' = 0.57$ and $\phi' = 30°$. Through O, Fig. 9, draw OQ, cutting the base of wall $5A$ at 6, and making an angle $\phi' = 30°$ with a vertical line through 6. Then the factor of safety against sliding will be

$$f_s = QM'/RM = 2.5$$

This is ample as the resistance of the filling in front of the toe will increase the resistance against sliding.

3. *Stability Against Crushing.*—In Fig. 9 the direct pressure will be $p_1 = 49{,}580/15.21 = 3{,}220$ lb. per sq. ft.

The pressure due to bending will be

$p_2 = \pm 6F \cdot b/d^2 = \pm (6 \times 49{,}580 \times 2.1)/231.4 = \pm 2{,}700$ lb. per sq. ft., and the maximum pressure is

$$p = 3{,}220 + 2{,}700 = +5{,}920 \text{ lb. per sq. ft.}$$

and the minimum pressure is

$$p = 3{,}220 - 2{,}700 = +\ 520 \text{ lb. per sq. ft.}$$

The allowable pressure was 6,000 lb. per sq. ft., so that the pressure is safe for a compact gravel. Where the walls were supported on the mushy clay it was necessary to extend the projection of the footing on the front side and to bring the resultant F to the center of the wall.

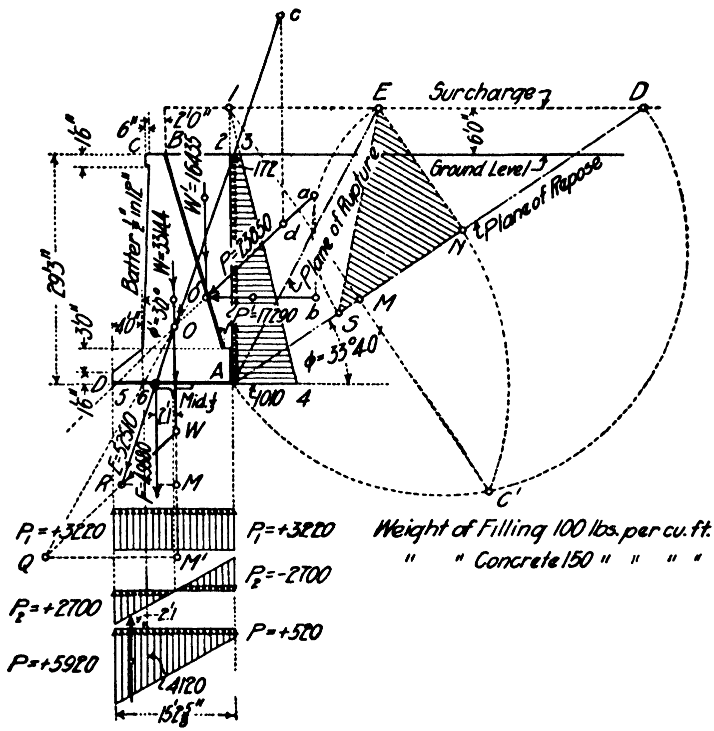


FIG. 9. RETAINING WALL, WEST ALAMEDA AVENUE SUBWAY.

4. *Upward Pressure on Front Projection of Foundation.*—Where projections are used on the foundations of retaining walls it may be necessary to reinforce the base to prevent the projection breaking off in line with the face of the wall. The bending moment of the upward pressure about the front face of the wall from Fig. 9 is

$$M = \tfrac{1}{2}(5{,}920 + 4{,}120) \times 4 \times 2.1 \times 12$$
$$= 506{,}000 \text{ in-lb.}$$

The tension on the concrete at the bottom of the footing will be

$$f = M \cdot c/I = M \cdot d/2I = (506{,}000 \times 27)/157{,}464$$
$$= 88 \text{ lb. per sq. in.}$$

Since the ultimate strength of the concrete in tension is approximately 200 lb. per sq. in.,

no reinforcing is required. However, ¾ in. □ bars were placed 18 in. centers and 3 in. from the bottom of the foundation.

Data.—The coefficients of friction of various materials are given in Table I. The angles of repose of different materials are given in Table II. The conditions of surface and amount of moisture cause wide variations in the coefficients. Additional data for the design of retaining walls are given in Tables III to VI.

TABLE I.

COEFFICIENTS OF FRICTION.

Materials.	Coefficients.	Materials.	Coefficients.
Dry masonry on dry masonry	0.6 to 0.7	Masonry on dry clay	0.5 to 0.6
Masonry on masonry with wet mortar	0.75	Masonry on moist clay	0.33
Timber on stone	0.4	Earth on earth	0.25 to 1.0
Iron on stone	0.3 to 0.7	Hard brick on hard brick	0.7
Timber on timber	0.2 to 0.5	Concrete blocks on concrete blocks	0.65

TABLE II.

ANGLES OF REPOSE, ϕ, FOR MATERIALS.

Materials.	ϕ	Materials.	ϕ
Earth, loam	30° to 45°	Clay	25° to 45°
Sand, dry	25° to 35°	Gravel	30° to 40°
Sand, moist	30° to 45°	Cinders	25° to 40°
Sand, wet	15° to 30°	Coke	30° to 45°

TABLE III.

ALLOWABLE PRESSURE ON FOUNDATIONS.

Material.	Pressure in Tons per Sq. Ft.
Soft clay	1 to 2
Ordinary clay and dry sand mixed with clay	2 to 3
Dry sand and clay	3 to 4
Hard clay and firm, coarse sand	4 to 6
Firm, coarse sand and gravel	6 to 8
Bed rock	15 and up.

TABLE IV.

ALLOWABLE PRESSURE ON MASONRY.

Materials.	Pressure in Tons per Sq. Ft.
Common brick, Portland cement mortar	12
Paving brick, Portland cement mortar	15
Rubble masonry, Portland cement mortar	12
Sandstone, first class masonry	20
Limestone, first class masonry	25
Granite, first class masonry	30
Portland cement concrete, 1-2-4	25
Portland cement concrete, 1-3-6	20

TABLE V.

WEIGHT, SPECIFIC GRAVITY AND CRUSHING STRENGTH OF MASONRY.

Materials.	Weight in Pounds per Cubic Foot.	Specific Gravity.	Crushing Strength in Pounds per Square Inch.
Sandstone	150	2.4	4,000 to 15,000
Limestone	160	2.6	6,000 to 20,000
Trap	180	2.9	19,000 to 33,000
Marble	165	2.7	8,000 to 20,000
Granite	165	2.7	8,000 to 20,000
Paving brick, Portland cement	150	2.4	2,000 to 6,000
Stone concrete, Portland cement	140 to 150	2.2 to 2.4	2,500 to 4,000
Cinder concrete, Portland cement	112	1.8	1,000 to 2,500

TABLE VI.

WEIGHT OF DIFFERENT MATERIALS.

Materials.	Wt. per Cu. Ft., Lb.	Materials.	Wt. per Cu. Ft., Lb.
Loam, loose	75 to 90	Sand, wet	110 to 120
Loam, rammed	90 to 100	Gravel	120 to 135
Sand, dry	90 to 110	Soft flowing mud	105 to 120

For specifications for concrete, plain and reinforced, see Chapter VI.

EXAMPLES OF RETAINING WALLS.—Details of six masonry retaining walls with a gravity section are given in Fig. 10. These retaining walls represent the best practice. Details of four reinforced concrete retaining walls are given in Fig. 11. For additional examples see the author's "The Design of Walls, Bins and Grain Elevators."

The contents of standard concrete retaining walls, as designed by the Illinois Central Railroad, are given in Fig. 12.

DESIGN OF RETAINING WALLS AND ABUTMENTS.*—The Committee believes that the intelligent use of theoretical formulas leads to economical and proper design, and therefore recommends that Rankine's formulas which consider that the filling is a granular mass of indefinite extent, without cohesion, be used in the design of retaining walls. It is recommended that retaining walls be designed (*a*) for a level surcharge, or (*b*) for a sloping surcharge at the angle of repose. or (*c*) for a level surcharge with a uniform surcharge loading. Formulas based on Rankine are given for vertical walls, walls leaning away from the filling, and for walls leaning toward the filling.

The use of a fixed ratio of width to height leads to a neglect of the distribution of the pressure on the foundation. This is a question of great importance, since it is well established that movements from the original alignment, due to unequal settlement, form a defect more common than any other. The Committee feels that attention should be called to the importance of making a study of each case in designing a wall, particularly of the weight and character of the filling, and the amount and distribution of the pressure on the bed of foundations.

DESIGN OF RETAINING WALLS.—The following nomenclature is recommended:

ϕ = the angle of repose of the filling.

θ = the angle between the back of the wall and a horizontal line passing through the heel of the wall and extending from the back into the fill.

δ = angle of surcharge, the angle between a horizontal line and the surface of the filling. (It is recommended that values of $\delta = 0$ or $\delta = \phi$ be used.)

λ = the angle between the resultant thrust, P, and a horizontal line.

h = vertical height of the wall in feet.

h' = height of surcharge in feet.

* Report of the masonry committee of American Railway Engineering Association, adopted March 22, 1917.

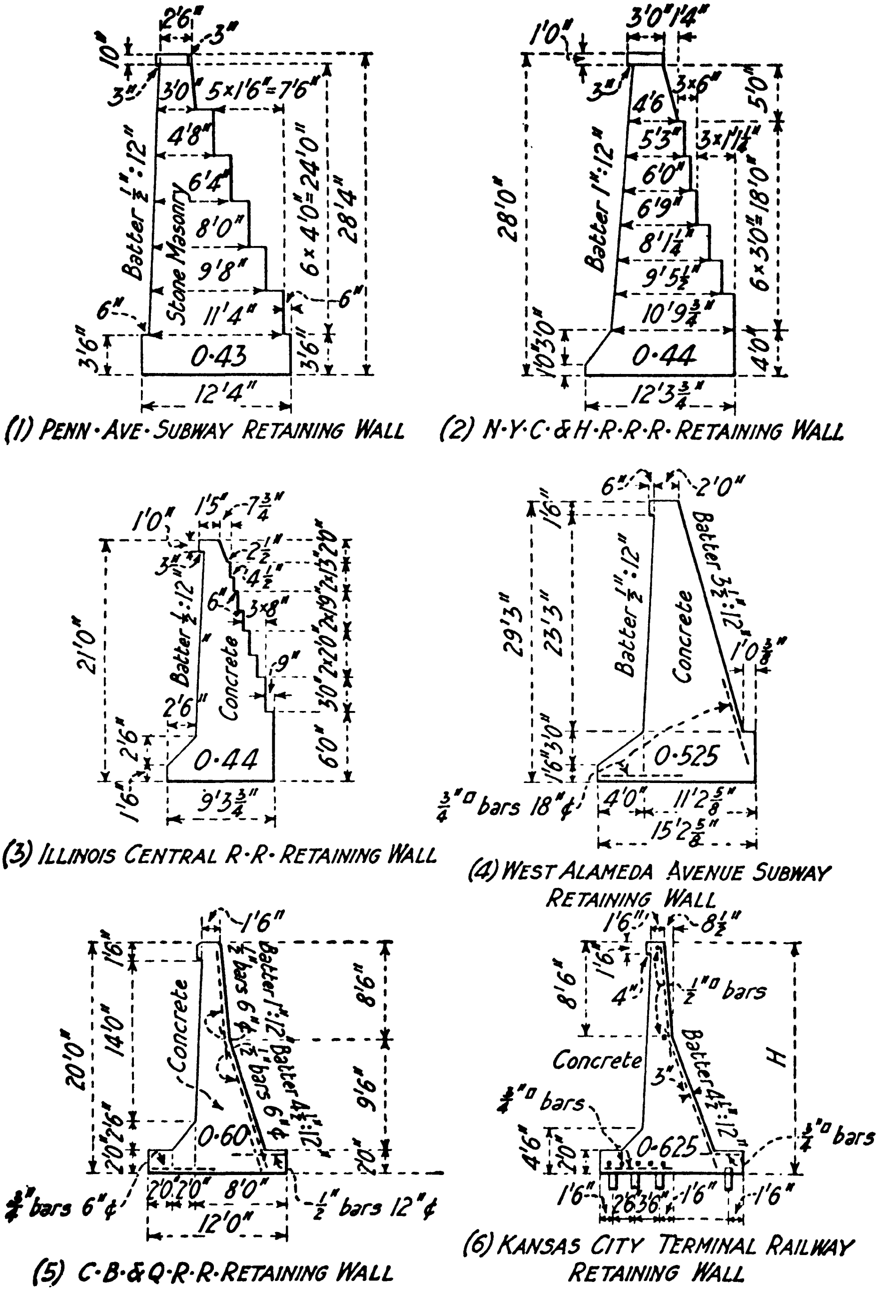


FIG. 10. EXAMPLES OF MASONRY RETAINING WALLS.

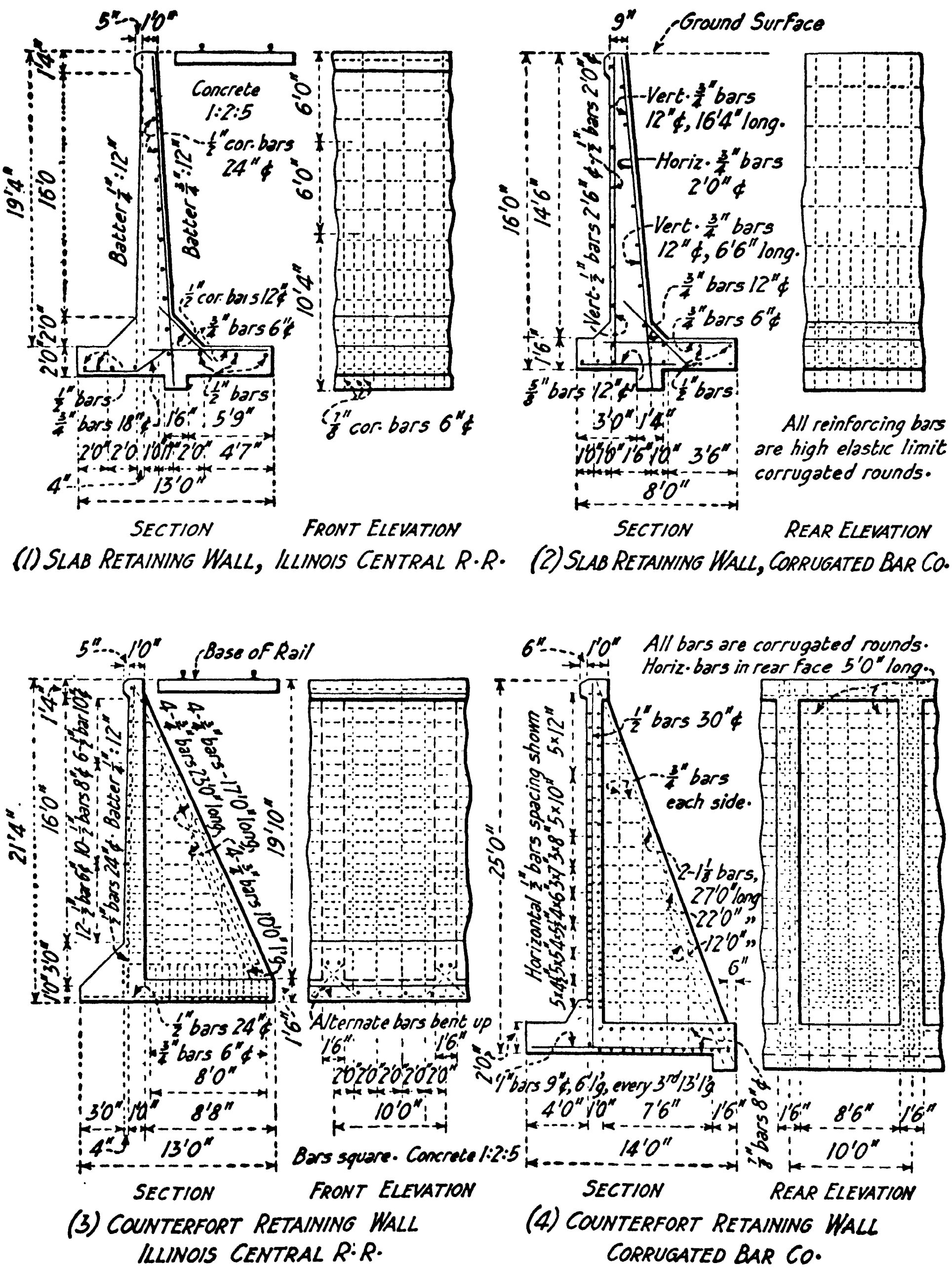


FIG. 11. EXAMPLES OF REINFORCED CONCRETE RETAINING WALLS.

l = width of the base of the wall in feet.
e = distance from the center of the base to the intersection of the resultant thrust, E, and the base.
$a = l/2 - e$ = distance from toe of wall to intersection of the resultant thrust, E, and the base.
P = the resultant earth pressure per foot of length of wall.
E = the resultant of the earth pressure and the weight of the wall.
F = vertical component of resultant E.
w = the weight of the filling per cubic foot.
w_1 = the weight of the masonry per foot of length.
W = total weight of the wall per foot of length.
p_1 and p_2 = pressure per square foot on the foundation, due to F, at toe and heel, respectively.

Formulas.—The following formulas for vertical walls or for walls leaning away from the filling are based on Rankine's Theory, as given in Howe's "Retaining Walls," and in Ketchum's "Walls, Bins and Grain Elevators"; and the formulas for walls leaning toward the filling are based on a modification of Rankine's Theory, as given in Ketchum's "Walls, Bins and Grain Elevators."

For vertical walls with horizontal surcharge the pressure, P, is given by the formula

$$P = \tfrac{1}{2}w \cdot h^2 \frac{1 - \sin\phi}{1 + \sin\phi} = \tfrac{1}{2}w \cdot h^2 \cdot \tan^2\left(45^\circ - \frac{\phi}{2}\right) \tag{32}$$

where P is parallel to the top surface, is normal to the wall, and is applied at one-third the height of the wall above the base.

For vertical walls with a positive surcharge, δ the pressure, P, is giv n by the formula

$$P = \tfrac{1}{2}w \cdot h^2 \cdot \cos\delta \frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}} \tag{33}$$

where P is parallel to the top surface of the filling, makes an angle δ with a normal to the back of the wall, and is applied at one-third the height of the wall above the base. Where the surcharge is equal to the angle of repose, ϕ, formula (33) becomes

$$P = \tfrac{1}{2}w \cdot h^2 \cdot \cos\phi \tag{34}$$

For a vertical wall with a loaded surcharge the resultant pressure on the back of the wall will be given by the formula

$$P = \tfrac{1}{2}w \cdot h(h + 2h') \frac{1 - \sin\phi}{1 + \sin\phi} \tag{35}$$

where h is the height of the wall and h' the equivalent height of surcharge, equals surcharge per square foot divided by w, the weight per cubic foot of the filling.

The resultant pressure is horizontal and is applied at a distance from the base of the wall equal to

$$y = \frac{h^2 + 3h \cdot h'}{3(h + 2h')} \tag{36}$$

(*a*) In calculating the surcharge due to a track the entire load shall be taken as distributed uniformly over a width of 14 feet for a single track or tracks spaced more than 14 feet centers, and the distance center to center of tracks where tracks are spaced less than 14 feet.

(*b*) In calculating the pressure on a retaining wall where the filling carries permanent tracks or structures, the full effect of the loaded surcharge shall be considered where the edge of the distributed load or the structure is vertically above the back edge of the heel of the wall. The effect of the loaded surcharge may be neglected where the edge of the distributed load or the structure is at a distance from the vertical line through the back edge of the heel of the wall equal to h,

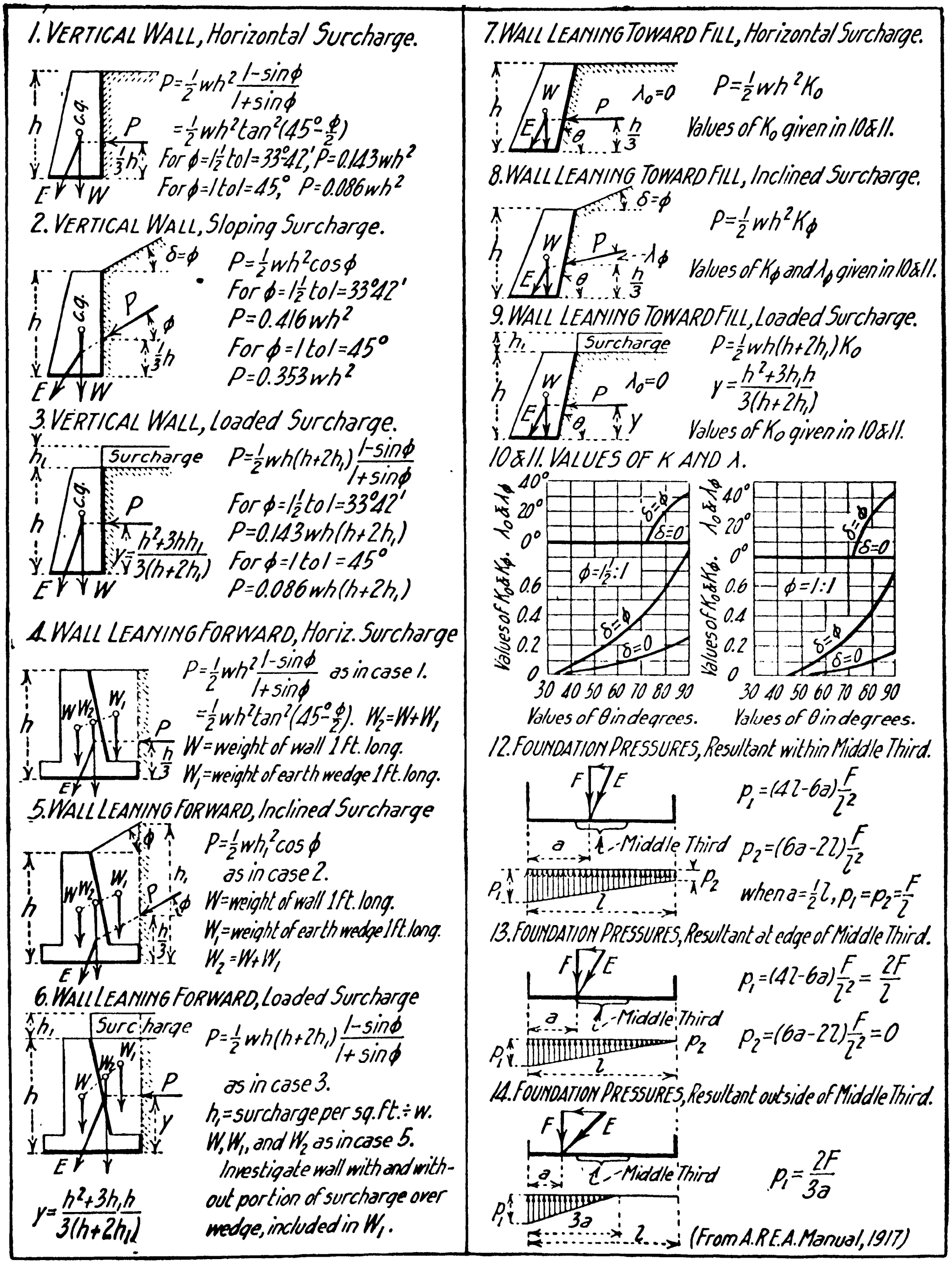

1. VERTICAL WALL, Horizontal Surcharge.
$P=\frac{1}{2}wh^2\frac{1-\sin\phi}{1+\sin\phi}$
$=\frac{1}{2}wh^2\tan^2(45°-\frac{\phi}{2})$
For $\phi=1\frac{1}{2}$ to 1 = 33°42', P = 0.143 wh^2
For ϕ = 1 to 1 = 45°, P = 0.086 wh^2
2. VERTICAL WALL, Sloping Surcharge.
$P=\frac{1}{2}wh^2\cos\phi$
For $\phi=1\frac{1}{2}$ to 1 = 33°42'
P = 0.416 wh^2
For ϕ = 1 to 1 = 45°
P = 0.353 wh^2
3. VERTICAL WALL, Loaded Surcharge.
$P=\frac{1}{2}wh(h+2h_1)\frac{1-\sin\phi}{1+\sin\phi}$
For $\phi=1\frac{1}{2}$ to 1 = 33°42'
P = 0.143 $wh(h+2h_1)$
For ϕ = 1 to 1 = 45°
P = 0.086 $wh(h+2h_1)$
$y=\frac{h^2+3hh_1}{3(h+2h_1)}$
4. WALL LEANING FORWARD, Horiz. Surcharge
$P=\frac{1}{2}wh^2\frac{1-\sin\phi}{1+\sin\phi}$ as in case 1.
$=\frac{1}{2}wh^2\tan^2(45°-\frac{\phi}{2})$. $W_2=W+W_1$
W = weight of wall 1 ft. long.
W_1 = weight of earth wedge 1 ft. long.
5. WALL LEANING FORWARD, Inclined Surcharge
$P=\frac{1}{2}wh_1^2\cos\phi$
as in case 2.
W = weight of wall 1 ft. long.
W_1 = weight of earth wedge 1 ft. long.
$W_2=W+W_1$
6. WALL LEANING FORWARD, Loaded Surcharge
$P=\frac{1}{2}wh(h+2h_1)\frac{1-\sin\phi}{1+\sin\phi}$
as in case 3.
h_1 = surcharge per sq. ft. ÷ w.
W, W_1, and W_2 as in case 5.
Investigate wall with and without portion of surcharge over wedge, included in W_1.
$y=\frac{h^2+3h_1h}{3(h+2h_1)}$
7. WALL LEANING TOWARD FILL, Horizontal Surcharge.
$P=\frac{1}{2}wh^2K_0$
$\lambda_0=0$
Values of K_0 given in 10 & 11.
8. WALL LEANING TOWARD FILL, Inclined Surcharge.
$\delta=\phi$
$P=\frac{1}{2}wh^2K_\phi$
Values of K_ϕ and λ_ϕ given in 10 & 11.
9. WALL LEANING TOWARD FILL, Loaded Surcharge.
$P=\frac{1}{2}wh(h+2h_1)K_0$
$\lambda_0=0$
$y=\frac{h^2+3h_1h}{3(h+2h_1)}$
Values of K_0 given in 10 & 11.
10 & 11. VALUES OF K AND λ.
$\phi=1\frac{1}{2}:1$
$\phi=1:1$
Values of K_0 & K_ϕ.
λ_0 & λ_ϕ
Values of θ in degrees.
12. FOUNDATION PRESSURES, Resultant within Middle Third.
$p_1=(4l-6a)\frac{F}{l^2}$
$p_2=(6a-2l)\frac{F}{l^2}$
when $a=\frac{1}{2}l$, $p_1=p_2=\frac{F}{l}$
Middle Third
13. FOUNDATION PRESSURES, Resultant at edge of Middle Third.
$p_1=(4l-6a)\frac{F}{l^2}=\frac{2F}{l}$
$p_2=(6a-2l)\frac{F}{l^2}=0$
14. FOUNDATION PRESSURES, Resultant outside of Middle Third.
$p_1=\frac{2F}{3a}$
(From A.R.E.A. Manual, 1917)

FIG. 12.

the height of the wall. For intermediate position the equivalent uniform surcharge load is to be taken as proportional. For example, for a track with the edge of the distributed load at a distance, $h/2$, from the vertical line through the back edge of the heel of the wall, the equivalent uniform surcharge load is one-half the normal distributed load distributed over the filling. Case 15, Fig. 13, explains the distribution. The height of surcharge loading will be equal to the load per linear foot divided by b ($b = 14$ feet for a single track railway). Where the edge of the distributed load cannot come nearer to the vertical line through the back of the heel of the wall than $h - x$, the equivalent uniformly distributed load in terms of heights is

$$h_x' = h'\frac{x}{h}.$$

For walls leaning forward or walls with the base extending into the filling, the pressure of the filling on a vertical plane through back of the heel of the wall, as calculated above, is to be combined with the wedge of filling contained between this vertical plane and the back of the wall.

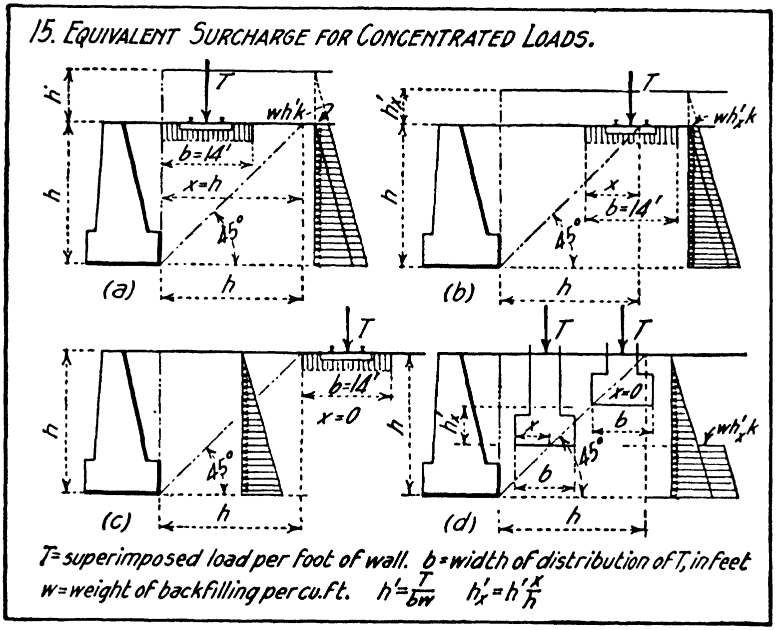


FIG. 13. EQUIVALENT SURCHARGE FOR CONCENTRATED LOADS.

For walls leaning toward the filling the resultant pressure, P, will be horizontal for a wall without surcharge or with a horizontal loaded surcharge, and will make an angle, λ, with the horizontal for a wall with a sloping surcharge. The values of λ will vary from δ, where the wall is vertical, to zero, where Rankine's Theory shows that the resultant pressure is horizontal. Values of λ are given in cases 10 and 11, Fig. 12. Values of K, where $P = \frac{1}{2}w \cdot h^2 \cdot K$, are given in cases 10 and 11, Fig. 12.

The formulas for the different cases above are given in cases 1 to 9, Fig. 12.

Discussion of Formulas.—Cases 1 to 3 are for vertical walls without heels. The pressure, P, is the same as the pressure on a vertical plane in the filling. Vertical walls with heels come under cases 4 to 6.

Cases 4 to 6 are for walls with heels. The wall may be vertical or may lean forward, or may lean backward, as long as the upper edge of the back of the wall is in front of the vertical plane through the edge of the heel.

Cases 7 to 9 are for walls without heels. Walls with heels come under cases 4 to 6 as long as the upper edge of the back of the wall is in front of the vertical plane through the edge of the heel; if the upper edge of the back of the wall extends back of the vertical plane through the edge of the heel, the problem can be solved by combining the solutions of cases 4 to 6 and 7 to 9.

Pressure on Foundations.—The pressures on foundations will be calculated by the following formulas:

Where a is equal to or greater than $l/3$.

Pressure at the toe

$$p_1 = (4l - 6a)\frac{F}{l^2} \tag{37}$$

Pressure at the heel is

$$p_2 = (6a - 2l)\frac{F}{l^2} \tag{38}$$

Where a is less than $l/3$, the pressure at the toe is

$$p_1 = \frac{2F}{3a} \tag{39}$$

Principles for Design of Retaining Walls.—The following principles should be observed in the design and construction of retaining walls.

1. For usual conditions of the filling use an angle of repose of $1\frac{1}{2}$ to 1 ($\phi = 33° 42'$). For dry sand or similar material, a slope of 1 to 1 ($\phi = 45°$) may be used.

2. The maximum pressure at the toe of the retaining wall should never exceed the safe bearing pressure on the material considered.

3. When the retaining wall rests on a compressible material, where settlement may be expected, the resultant thrust, E, should strike at the middle or back of the middle of the base of the wall so that the wall will settle toward the filling (a = or $> l/2$).

4. When the retaining wall rests on a material where settlement may not be expected the resultant thrust, E, should not strike outside the middle third of the base (a = or $> l/3$), except as noted in (5) below.

5. Where the retaining wall rests on solid rock or is carried on piles the resultant thrust, E, may strike slightly outside the middle third, provided the wall is safe against overturning, and also provided the maximum allowable pressure is not exceeded.

6. In order that the retaining wall may be safe against sliding, the frictional resistance of the base, combined with the abutting resistance of the earth in front of the wall, must be greater than the horizontal thrust on the back of the wall.

7. The filling back of the wall should be carefully drained so that the wall may not be subjected to hydrostatic pressure.

8. The foundation for a retaining wall should always be placed below frost line.

9. A careful study should be made of the conditions in the design of each wall, and it should be remembered that no theoretical formulas can be more than an aid to the judgment of the experienced designer. The main value of theoretical formulas is in obtaining economical proportions, in obtaining a proper distribution of the stresses, and in making experience already gained more valuable.

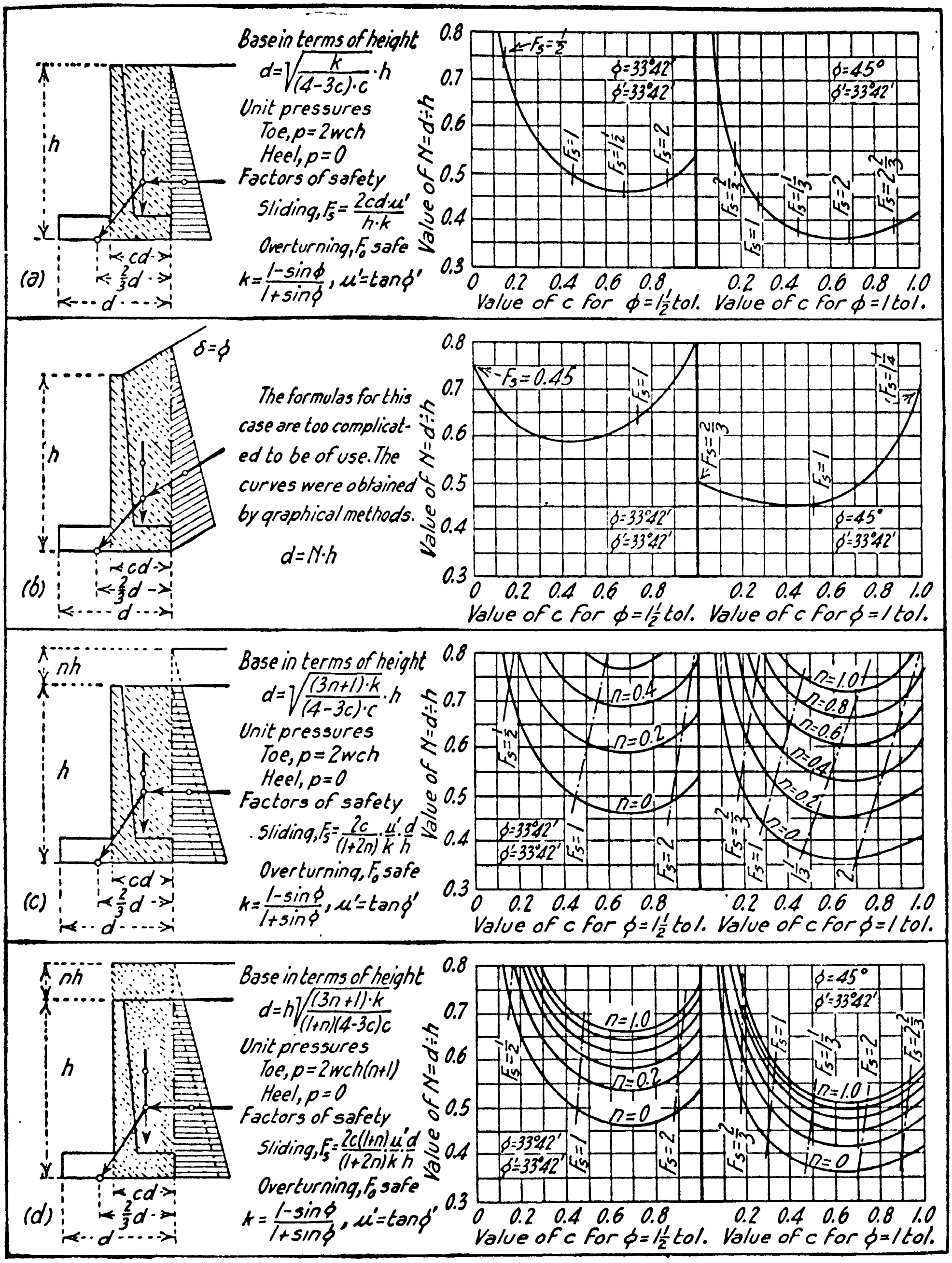


FIG. 14. APPROXIMATE DESIGN OF REINFORCED CONCRETE RETAINING WALLS. (Resultant thrust at outer edge of middle third of base.)

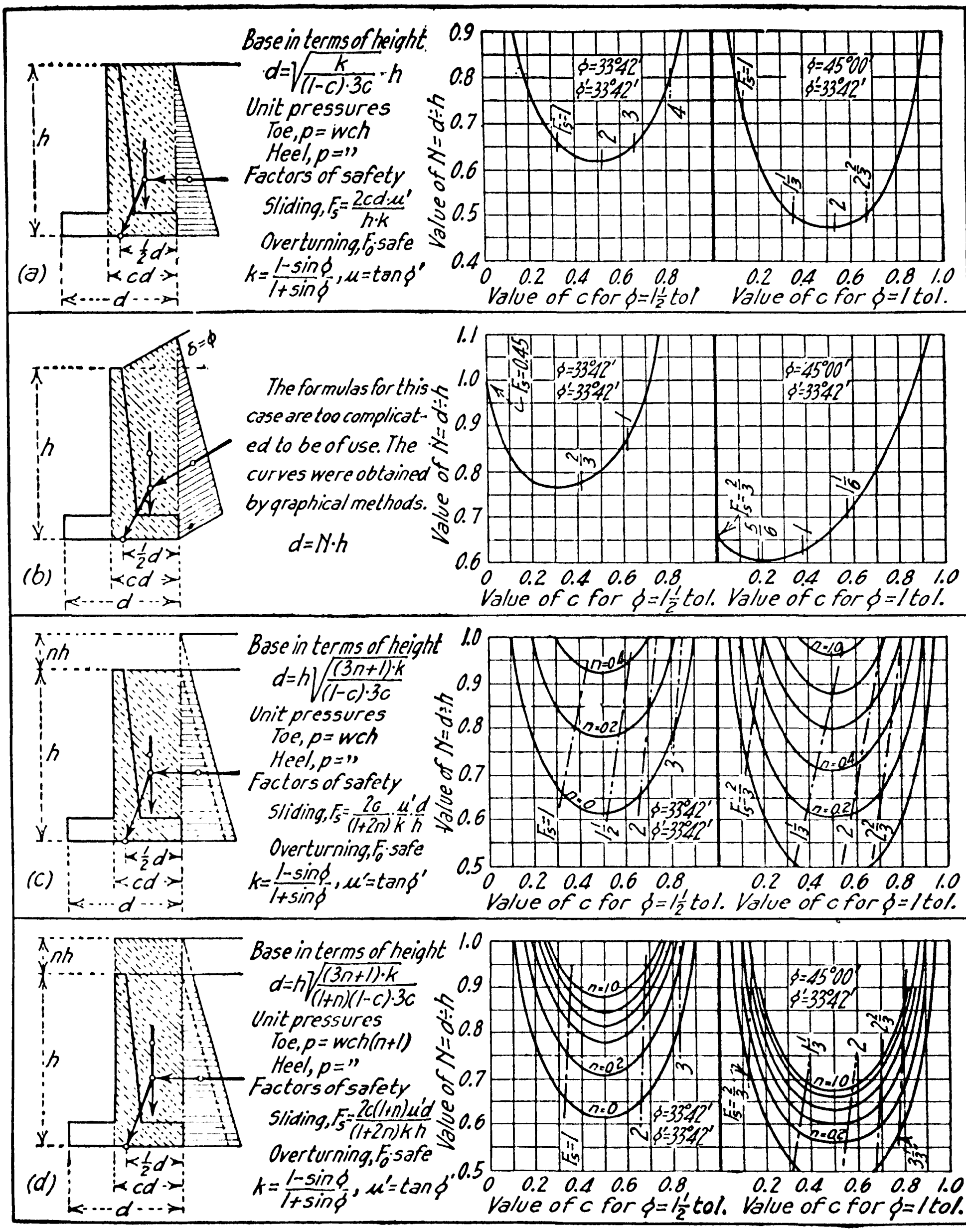


FIG. 15. APPROXIMATE DESIGN OF REINFORCED CONCRETE RETAINING WALLS. (Resultant thrust at center of base.)

APPROXIMATE DESIGN OF REINFORCED CONCRETE RETAINING WALLS.—The formulas and curves shown in Figs. 14 and 15 give the approximate dimensions, foundation pressures, and factors of safety for most types of reinforced concrete retaining walls, including the cantilever wall, and the counterfort wall.

These formulas and curves are based on the assumption that the wall and the filling have the same weight per cubic foot, and the weight of the toe is neglected. This is shown by the shaded areas in the figures. The values given by the formulas and curves are sufficiently accurate for determining the general dimensions of a wall preliminary to design.

Two sets of formulas are given. In Fig. 14 the resultant pressure on the foundation passes through the outside edge of the middle third of the base and in Fig. 15 the resultant pressure passes through the center of the base. See Principles of Design in this chapter.

Four cases have been considered: (*a*) the ground surface horizontal; (*b*) the ground surface sloping at the angle of repose; (*c*) a loaded surcharge up to a point vertically over the edge of the heel but not extending over the heel; and (*d*) a loaded surcharge extending over the heel. The loaded surcharge is taken as equivalent to some vertical load, such as loaded tracks, bins, etc.

The factor of safety against sliding is given by the nearly vertical lines crossing the solid curves. The angle of friction is taken as $\phi' = 33° 42'$ ($1\frac{1}{2}$ to 1) in all cases. The effect of a cut-off was not considered. The factor of safety against overturning is always safe in this type of wall.

Two sets of curves are given for each case, one with the angle of repose as $\phi = 33° 42'$ ($1\frac{1}{2}$ to 1) and one with the angle of repose as $\phi = 45° 00'$ (1 to 1). See Principles of Design in this chapter.

The curves show that, except in the case with sloping surcharge, the smallest width of base will be obtained when the front of the vertical slab is at a distance of two-thirds the width of the base from the heel when the resultant cuts the base at the edge of the middle third, and when the front of the vertical slab is at the center of the base when the resultant cuts the base at the center.

Concrete Retaining Walls. Methods of Constructing Forms.—Forms for a retaining wall may be built in sections, or may be built up each time they are used. The former method is much the cheaper, especially for plain concrete walls where the sections between expansion joints are of equal length. The forms used on the C. B. & Q. R. R. walls shown in Fig. 17 are shown in Fig. 18. The studs, coping and bottom forms for the face, and the back forming are sectional, while ordinary sheeting is used between the coping and bottom forms. No attempt was made to use sectional forms on the face of the wall, because the sections soon become badly warped, making a rough wall. The concrete had a tendency to lift the forms and they were tied to bars imbedded in the footings as shown. The sectional forms were 12 ft. 0 in. long, while the studs were spaced 3 ft. 0 in. center to center.

The forms for the Illinois Central R. R. retaining wall shown in Fig. 10 are shown in Fig. 15. The forms were built in sections 54 ft. long. The forms were cross-braced by $\frac{3}{4}$ in. rods spaced 7 ft. $8\frac{1}{2}$ in. center to center as shown. When the forms were taken down the ends of these rods were unscrewed, the main portion of the rod being left in the wall. The forms were made of 2 in. plank surfaced on the inside.

The forms used by the Chicago and Northwestern Ry. on track elevation in Chicago are shown in Fig. 20. The forms were built in sections 35 ft. long. The 2 in. × 8 in. braces were used to hold the sides of the forms apart and were removed as the concrete was put in place. The 2 in. pipe used to cover the rod bracing was old boiler flues and rejected pipe.

Ingredients in Concrete.—The proportions of concrete materials should be stated in terms of the volume of the cement. The volume of one barrel or four bags of cement is taken as 3.8 cu. ft., and the sand and aggregate are measured loose. Concrete mixed one part cement, 2 parts sand, and 4 parts stone is commonly called 1 : 2 : 4 concrete. The proportions should be such

that there should be more than enough cement paste to fill the voids in the sand, and more than enough mortar to fill the voids in the stone. With voids in sand and stone varying from 40 to 45 per cent, the quantities of the ingredients are closely given by Fuller's rule, where

c = number of parts of cement;
s = number of parts of sand;
g = number of parts of gravel or stone.

Then $\frac{11}{c+s+g} = p$ = number of barrels of Portland cement required for one cu. yd. concrete.

$\frac{p \times s \times 3.8}{27}$ = number of cu. yd. sand required for one cu. yd. concrete.

$\frac{p \times g \times 3.8}{27}$ = number of cu. yd. gravel or stone required for one cu. yd. concrete.

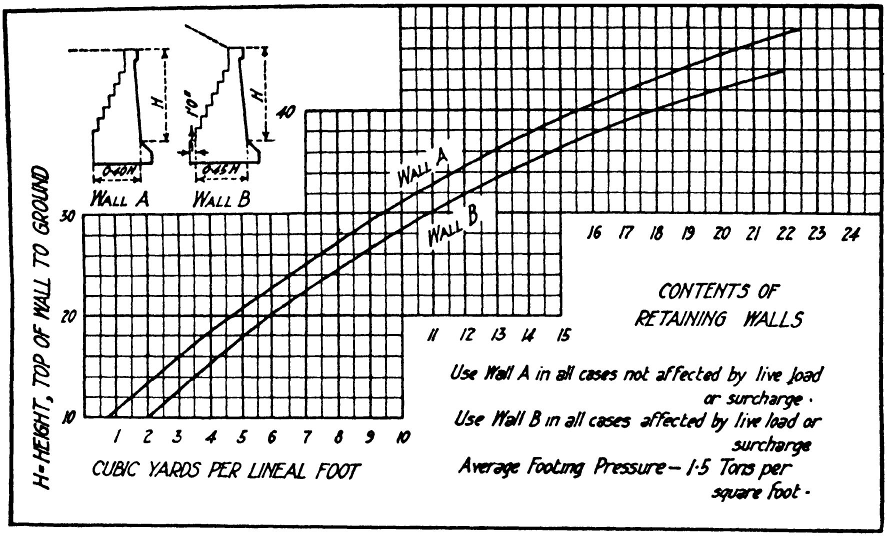


FIG. 16. CONTENTS OF CONCRETE RETAINING WALLS, ILLINOIS CENTRAL RAILROAD.

The materials for one cu. yd. of 1 : 2 : 4 concrete will then be: Portland cement 1.57 barrels, sand 0.44 cu. yd., gravel or stone 0.88 cu. yd.

The proportions for plain walls commonly vary from 1 : 2½ : 5 to 1 : 3 : 6, while the proportions for reinforced walls vary from 1 : 2 : 4 to 1 : 2½ : 5.

Mixing and Placing Concrete.—For mixing concrete a batch mixer in which the materials can be definitely proportioned and thoroughly mixed is to be preferred. In cold weather the concrete materials should be heated by the addition of boiling water to the mixer. To prevent scalding the cement the sand, aggregate and hot water should first be placed in the mixer and, after giving it several turns to remove the frost, the cement should be added and the mixing completed.

The author uses the following specifications for placing concrete in cold or freezing weather. "When the temperature of the air during the time of mixing and placing is below 40° Fah. the water used in mixing the concrete shall be heated to such a temperature, that the temperature of the concrete when deposited in the forms shall not be less than 60° Fah. Care shall be used not to scald the cement."

Where the wall is in a cut and the materials can be delivered on the bank, the mixer may be installed on the bank above and the concrete wheeled or chuted to place. Concrete should not be chuted in freezing weather. In building the West Alameda Avenue Subway retaining walls,

Denver, Colo., the gravel and sand were taken from the cut, the concrete was mixed in mixers installed at the foot of movable towers, and the concrete was raised in a skip elevator and chuted into place.

On railroad work the mixer may be mounted on a flat car, the materials may be delivered on other cars, and the concrete is dumped or chuted directly into place.

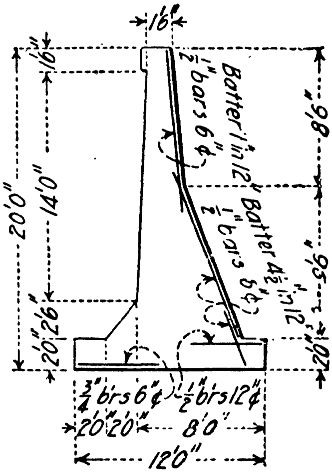


FIG. 17. RETAINING WALL, C. B. & Q. R. R.

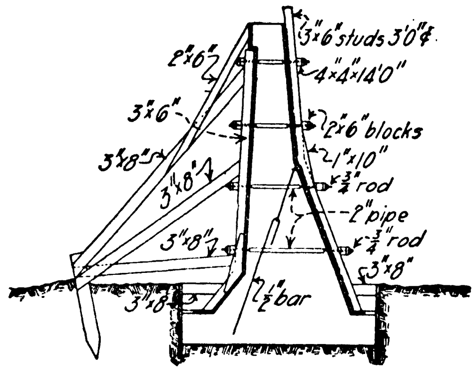


FIG. 18. FORMS FOR RETAINING WALL, C. B. & Q. R. R.

SPECIFICATIONS FOR CONCRETE RETAINING WALLS.—The following extracts have been taken from the specifications prepared by Crocker and Ketchum, Consulting Engineers, for the concrete retaining walls for the West Alameda Avenue Subway, Denver, Colo.

16. **MATERIALS. Cement.**—The cement shall be furnished by the Companies on board cars or in store houses at the site of the work as required. The cement shall be Portland, and shall meet the requirements of the Standard Specifications of the American Society for Testing Materials.

17. **Concrete Aggregate.**—The fine aggregate shall pass a screen with ¼ in. mesh, while the coarse aggregate shall all be retained on a screen with ¼ in. mesh and all shall pass a screen with 3 in. mesh. The sand and gravel shall be obtained from the excavation of the open cut of the Subway. The Consulting Engineers reserve the right to change the proportions of sand and screened gravel (§ 34 and § 35) from time to time, as may be necessary to secure a dense concrete of desired consistency. Payment to the Contractor for the screening will be made on the basis of unit price per cubic yard of gravel measured after screening.

18. **Water.**—The water used in mixing concrete shall be clean and reasonably clear, free from acids and injurious oils, alkalies or vegetable matter.

19. **Lumber.**—Lumber for forms shall have a nominal thickness of 2″ before surfacing, and shall be of a good quality of Douglas fir or Southern long leaf yellow pine. Lumber used for forms of face work shall be dressed on one side and both edges to a uniform thickness and width. Lumber for backing and other rough work may be unsurfaced and of an inferior grade of the kinds above specified.

20. **Reinforcing Steel.**—All reinforcing steel shall be plain bars, and shall comply with the specifications for structural steel as given in the Standard Specifications of the American Railway Engineering Association.

21. **EXCAVATION.**—The subway is being excavated by the Companies but the contractor shall make all necessary excavations for wall and pedestal footings, and shall furnish all necessary sheeting and supports and bracing to hold the forms in place during the construction of the work.

The cost of the necessary sheeting and supports shall be included in the unit price for excavation. The Contractor shall provide all pumps and other equipment incidental to such excavation.

22. All excavation shall be measured in vertical prisms whose end areas are of sufficient size to include the footing courses, and the sheeting surrounding the same. "Wet excavation" shall include all excavation below the surface of standing water in open pits.

23. **CONCRETE. Machine Mixing.**—Machine mixers, preferably of the batch type, shall be used except where the volume of concrete to be mixed is not sufficient to warrant their use. The requirements are that the product delivered shall be of the specified proportions and consistency, and thoroughly mixed.

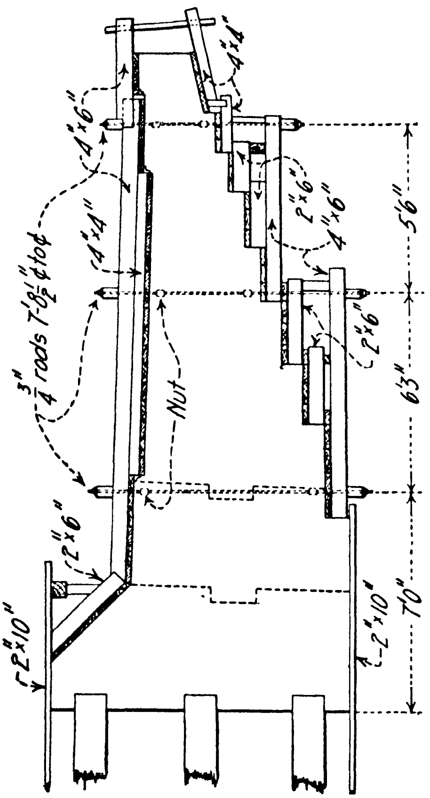


FIG. 19. FORMS FOR ILLINOIS CENTRAL R. R. RETAINING WALL.

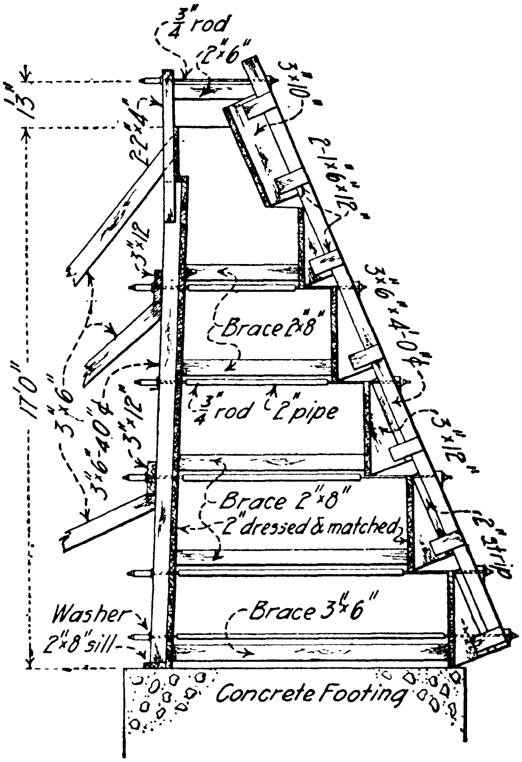


FIG. 20. FORMS FOR C. & N. W. RY. RETAINING WALL.

24. **Mixing by Hand.**—When it is necessary to mix by hand the mixing shall be done on water tight platforms of sufficient size to accommodate men and materials for the progressive and rapid mixing of at least two batches of concrete at the same time. Batches shall not exceed one-half yard. The mixing shall be done as follows: The fine aggregate shall be spread evenly upon the platform, then the cement upon the fine aggregate and these mixed thoroughly until of an even color. Then add the coarse aggregate which, if dry, shall first be thoroughly wet down. The mass shall then be turned with shovels until thoroughly mixed and all the aggregate covered with mortar, the necessary amount of water being added as the mixing proceeds.

25. **Consistency.**—The material shall be mixed wet enough to produce a concrete of such consistency that it will flow into the forms and about the metal reinforcement, and which on the other hand can be conveyed from the place of mixing to the forms without the separation of the coarse aggregate from the mortar.

26. **Retempering.**—Retempering mortar or concrete, i. e.. remixing with water after it has partially set will not be permitted.

27. **Placing of Concrete.**—Concrete after the addition of water to the mix, shall be handled rapidly from the place of mixing to the place of final deposit, and under no circumstances shall concrete be used that has partially set before final placing.

28. The concrete shall be deposited in such a manner as will prevent the separation of the ingredients and permit the most thorough compacting. It shall be compacted by working with a straight shovel or slicing tool kept moving up and down until all the ingredients have settled in their proper place, and the surplus water is forced to the surface. All concrete must be deposited in horizontal layers of uniform thickness throughout. Temporary planking shall be placed at ends of partial layers so that the concrete shall not run out to a thin edge. In placing concrete it shall not be dropped through a clear space of over 6 ft. vertical. For greater heights a trough or other suitable device must be used to deliver the concrete in place, and in depositing each batch this trough or other device must first be carefully filled with concrete and then as fast as concrete is removed at the bottom it shall be replenished at the top.

29. The work shall be carried up in alternate sections of approximately 32 ft. in length as shown on the plans, and each section shall be completed without intermission. In no case shall work on a section stop within 18 in. of the top.

30. Before depositing concrete, the forms shall be thoroughly wetted, except in freezing weather, and the space to be occupied by the concrete cleared of debris.

31. **Expansion Joints.**—Expansion joints shall be provided (sections were approximately 32 ft. long) as shown on the plans. The wall shall be constructed in alternate sections, the ends of the sections being formed by vertical end forms, the section being completed as though it were the end of the structure. Before placing the remaining sections the end forms shall be removed and the surface of the concrete shall be painted with coal tar paint, composed of sixteen (16) parts coal tar, four (4) parts Portland cement and three (3) parts kerosene oil. The expansion joints shall be finished on the exposed side by the insertion in the forms of a metal mold that will give a groove ½ in. wide, 1 in. deep and shall have a draft of 1 in. The wall sections shall be locked together by means of bars as shown on the plans.

32. **Forms.**—Forms shall be substantial and unyielding and built so that the concrete shall conform to the design, dimensions and contours, and so constructed as to prevent the leakage of mortar. Where corners of the masonry and other projections liable to injury occur, suitable moldings shall be placed in the angles of the forms to round or bevel them off. Material once used in forms shall be cleaned before being used again.

33. The forms must not be removed within 36 hours after all the concrete in that section has been placed; in freezing weather they must remain until the concrete has had sufficient time to become thoroughly set.

34. **Proportioning.**—In proportioning concrete, a barrel or 4 sacks of Portland cement shall be assumed to contain 3.8 cu. ft., while the sand and gravel shall be measured loose in a measuring vessel. The proportions required for concrete are as follows:

For footings, walls of retaining walls, abutments, and pedestals, one (1) part Portland cement, three (3) parts sand and five (5) parts gravel. For bridge seats and copings, one (1) part Portland cement, two (2) parts sand and four (4) parts gravel.

35. The tops of the bridge seats, pedestals, and copings, shall be finished with a smooth surface composed of one (1) part Portland cement and two (2) parts sand applied in a layer 1 in. thick. This must be put in place with the last course of concrete.

36. **Water-Proofing.**—The expansion joints in the retaining walls and abutments shall be water-proofed as follows: After the forms have been removed and the concrete is thoroughly dried, the back of the wall for a distance of 18 in. on each side of the expansion joints shall be mopped with hot refined coal tar pitch. A layer of burlap shall then be placed so as to cover the expansion joints, and the burlap shall be mopped with coal tar pitch. In the same manner two additional layers of burlap shall be applied, making a 3-ply water-proofing.

37. **Reinforcing Bars.**—Reinforcing bars, where used, shall be placed 3 in. clear from the outside surface of the concrete, and shall be placed in the position shown on the plans. Care must be taken to insure the coating of the metal with mortar, and a thorough compacting of concrete around the bars. All reinforcing bars shall be clean and free from all dirt or grease.

38. **Freezing Weather.**—Concrete shall not be mixed or deposited at a freezing temperature, unless special precautions are taken to avoid the use of materials containing frost or covered with ice, and means are provided to prevent the concrete from freezing. Where the temperature of the air during the time of mixing and placing concrete is below 40° Fahr. the water used in mixing the concrete shall be of such a temperature, that the temperature of the concrete when delivered in the forms shall not be lower than 60° Fahr. Special precautions shall be taken not to scald the cement.

39. **Placing in Water.**—Concrete shall not be deposited under water except on the approval of the Consulting Engineers. Where water is encountered without current, but in such quantity that it cannot be lowered to the required depth and maintained there, or where such lowering

would cause further difficulty, concrete may be deposited through troughs or other device in the manner designated above.

40. **Cleaning Up.**—Upon the completion of any section of the work the Contractor shall remove all debris caused by his operations and leave the work ready for backfilling.

REFERENCES.—For the design of reinforced concrete retaining walls, examples of plain and reinforced concrete retaining walls, details of construction, and the theory of reinforced concrete, see the author's "The Design of Walls, Bins and Grain Elevators." For a discussion of the theory of the pressures in granular materials and semi-fluids, see Chapter VIII, Bins, and Chapter IX, Grain Elevators; also see the author's "The Design of Walls, Bins and Grain Elevators."

CHAPTER VI.

BRIDGE ABUTMENTS AND PIERS.

Introduction.—An abutment is a structure that supports one end of a bridge span and at the same time supports the embankment that carries the track or roadway. An abutment also usually protects the embankment from the scour of the stream.

A pier is a structure that supports the ends of two bridge spans. Piers must be designed so as not to interfere with the flow of the stream, and care must be used to prevent undermining the pier by the scour of the stream.

TYPES OF ABUTMENTS.—Masonry abutments may be classified under four heads, Fig. 1, (*a*) straight or "stub" abutments; (*b*) wing abutments; (*c*) U abutments; (*d*) T abutments.

(*a*) The standard straight abutment of the N. Y. C. & H. R. R. R., shown in Fig. 1, is an excellent example of an abutment of this type. The earth fill is allowed to flow around the ends of the abutment as shown. Straight abutments should not be used where the water will wash the fill away.

(*b*) A standard wing abutment of the N. Y. C. & H. R. R. R. is shown in Fig. 1. The length of the wings is determined by the width of the roadway, the allowable slope of the sides of the embankment and the angle of the wings. The angle that the wings make with the face of the abutment ordinarily varies from 30 degrees to 45 degrees for standard conditions. For skew bridges and for unusual conditions the angle of the wing is variable.

(*c*) A standard U abutment of the N. Y. C. & H. R. R. R. is shown in Fig. 1. This is a wing abutment with the wings making an angle of 90 degrees with the face of the abutment. The wings are tied together by means of old railroad rails as shown. The wing walls run back into the fill, which flows down in front of the wings. If the slope is liable to be washed away by the scour of the stream the wings should be extended farther into the bank.

(*d*) A standard T abutment of the South Bend and Michigan Southern Railway for a skew span is shown in Fig. 1. The T abutment is essentially a straight abutment with a stem running back into the fill; the stem carries the roadway, supports the abutment, and prevents water from finding its way along the back of the abutment. A T abutment may be considered as a U abutment with the two wings in one.

STABILITY OF BRIDGE ABUTMENTS WITHOUT WINGS.—A bridge abutment must be stable (1) against overturning, (2) against sliding, and (3) against crushing the material on which the abutment rests, or the masonry in the abutment. The problem of the design of a bridge abutment is essentially the same as the design of a retaining wall, for which see Chapter V. The method of design will be shown by giving the calculations for a straight concrete abutment for West Alameda Avenue Subway, Denver, Colo.

Design of Concrete Abutment for West Alameda Avenue Subway, Denver, Colorado.—The height of the abutment is 21 ft. 6 in. from the bottom of the footing to the top of the bridge seat, and 25 ft. $0\frac{3}{8}$ in. to the top of the back wall. The following assumptions were made: Weight of concrete, 150 lb. per cu. ft.; weight of filling, w = 100 lb. per cu. ft.; angle of repose of the filling, $1\frac{1}{2}$ to 1 ($\phi = 33° 42'$); surcharge 800 lb. per sq. ft., equivalent to 8 ft. of filling; maximum load on foundation 6,000 lb. per sq. ft.

Solution.—After several trials the dimensions given in Fig. 2 were taken. The stability of the abutment was investigated for two conditions: (*a*) with a full live and dead load on the bridge and on the filling, and (*b*) with no live load on the bridge and no surcharge coming on the filling above the wall, it being assumed that a locomotive is approaching the bridge from the right, and

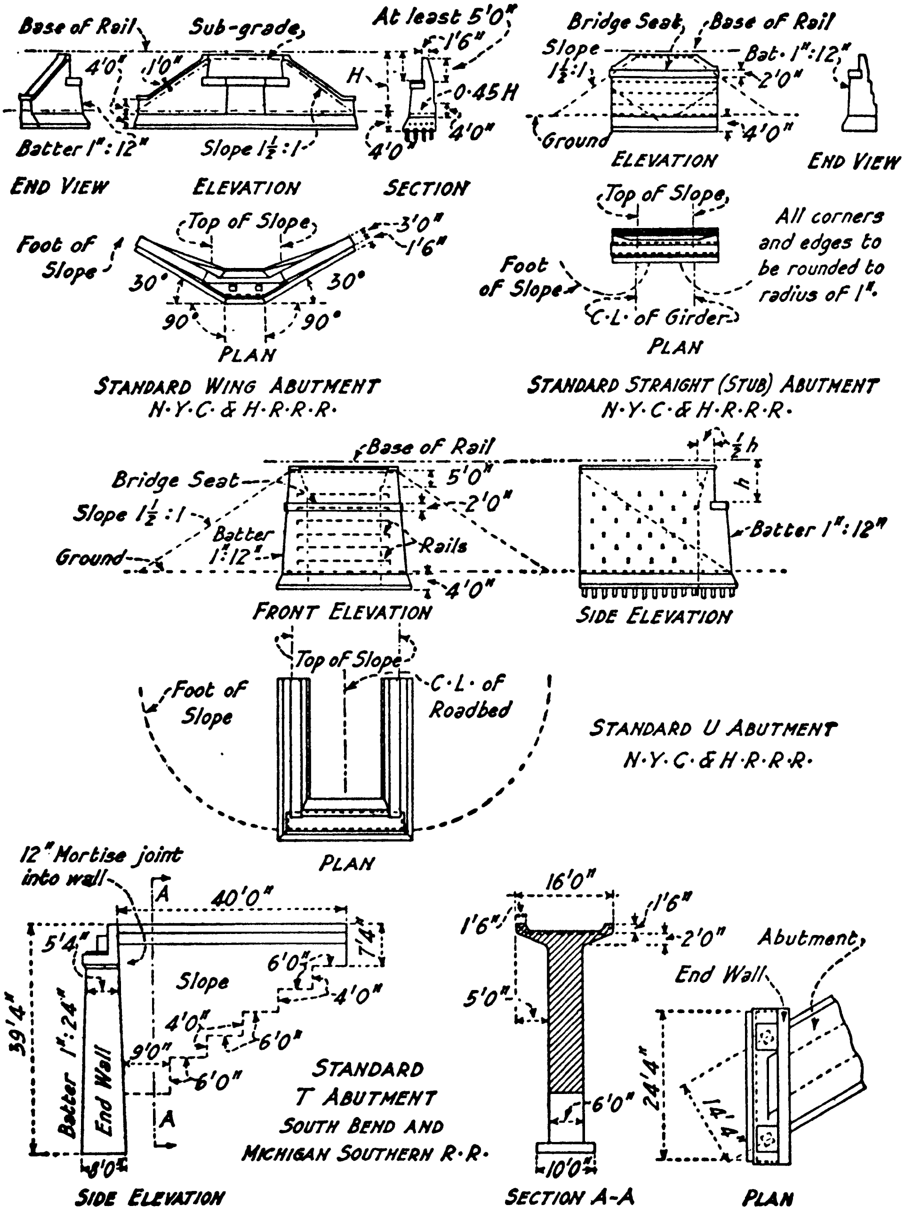


FIG. 1. TYPES OF MASONRY ABUTMENTS.

has reached the point 2 in (*b*), Fig. 2. The weight of the girders and the live load was assumed as uniformly distributed over a length of the abutment equal to the distance between track centers, and one lineal foot of wall was investigated.

Case (a).—The pressure of the filling on the plane B–2 was calculated as in Chapter V, Fig. 9, and is $P' = 14{,}700$ lb., acting through the center of gravity of the trapezoid 2–3–4–B. The weight of the filling and surcharge is $W_2 + W_3 = 14{,}900$ lb., which when combined with P' gives the resultant pressure of the filling on the wall $= P = 20{,}900$ lb. The pressure P is then combined with the weight of the wall, $W_1 = 29{,}800$ lb., and with the dead load and live load from the girder $= 12{,}820$ lb., giving the resultant pressure on the foundation, $E = 59{,}400$ lb., and acting, $b = 1.4$ ft. from the center of the wall, and $F = 57{,}500$ lb.

1. *Stability Against Overturning.*—The resultant E is nearly vertical and well within the middle third, so that the wall is amply safe against overturning.

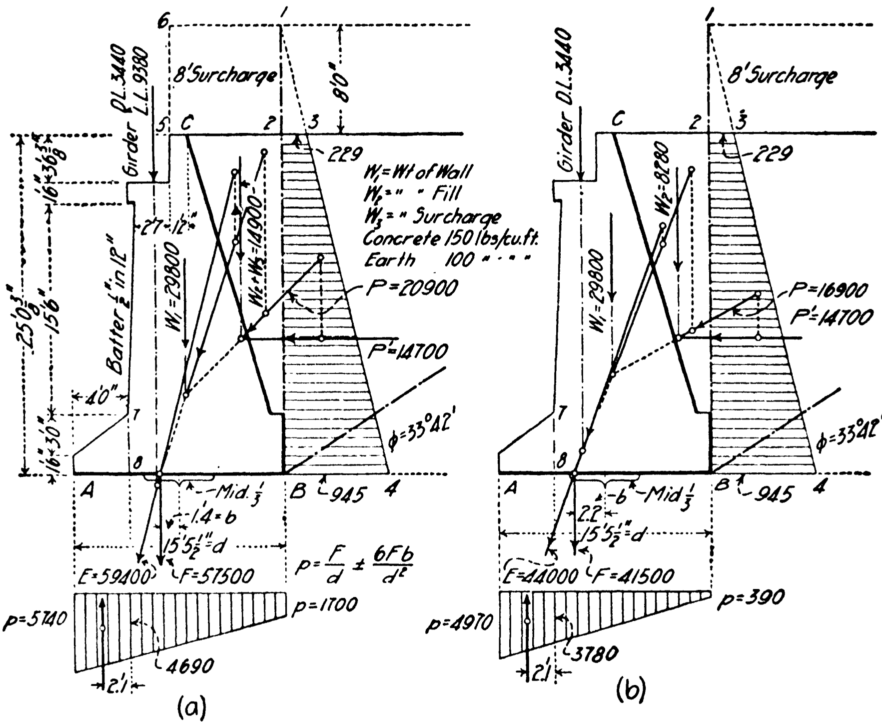


FIG. 2. ABUTMENT FOR WEST ALAMEDA AVENUE SUBWAY, DENVER, COLO.

2. *Stability Against Sliding.*—Assuming that $\phi' = 30°$, then the coefficient of friction will be $\tan \phi' = 0.57$. Using the definition of factor of safety given in equation (27) Chapter V, the resistance of the wall against sliding will be $57{,}500 \times 0.57 = 32{,}765$ lb. The sliding force is $P' = 14{,}700$ lb., and the factor of safety is $32{,}765/14{,}700 = 2.23$, which is ample.

3. *Pressure on Foundation.*—The pressure on the foundation will be $p = F/d \pm 6F \cdot b/d^2 = +5{,}740$ and $+1{,}700$ lb. per sq. ft., which is safe.

4. *Upward Pressure on Front Projection of Foundation.*—The base will be investigated on the plane 7–8 to see that the upward pressure will not break off the front projection of the foundation. The bending moment of the upward pressure about the front face of the wall in (*a*), Fig. 2, will be

$$M = \tfrac{1}{2}(5,740 + 4,690)4 \times 2.1 \times 12$$
$$= 525,672 \text{ in-lb.}$$

The tension on the concrete at the bottom of the footing will be

$$f = \frac{M \cdot c}{I} = \frac{M \cdot d}{2I} = \frac{525,672 \times 27}{157,464}$$
$$= 92 \text{ lb. per sq. in.}$$

The footing is safe, but ¾ in. □ rods were placed 18 in. centers and 3 in. from the bottom of the foundation.

Case (b).—The solution is the same as for (*a*) except that the live load from the girder = 9,980 lb., and the surcharge load 1-2-5-6 = W_3 = 6,620 lb. were omitted. The wall is safe for overturning. The factor of safety against sliding is from equation (27) Chapter V, $f_s = 41,500 \times 0.57/14,700 = 1.6$, which is safe. The pressure on the foundation is safe.

The back wall was placed after the bridge seats were finished. To bond the back wall to the abutment, ½ in. □ rods 4 ft. long, spaced 18 in. centers, were placed in two rows 3 in. from the back and front face, one-half of the length of the rod being imbedded in the main wall.

PRINCIPLES OF DESIGN.—To prevent tension on the back side of the footing and to make sure that the maximum compression on the front side of the footing shall not be greater than twice the average pressure, the resultant of the thrust of the filling, the weight of the masonry, the weight of the bridge and the live load must strike within the middle third of the base. Where the abutment rests on rock or solid material where settlement will not occur, it will not be serious if the resultant strikes a little outside of the middle third, providing the allowable pressure on the foundation is not exceeded. When the abutment is on compressible material where settlement will take place. the resultant of the pressures should strike at or back of the center of the base, so that the abutment will not tip forward in settling. It is standard practice to use piles in the foundation for abutments resting on compressible soil.

For the design of wing walls see the design of Retaining Walls, Chapter V.

In addition to the requirements for stability abutments should satisfy the following additional requirements.

(*a*) The abutment should protect the bank from scour. (*b*) The abutment should prevent the embankment drainage from washing away the bank. (*c*) The abutment should be easily drained.

Empirical Design.—A common rule is to make the minimum thickness of the main part of the abutment not less than $\frac{4}{10}$ the height above any section; and project the footings on each side as may be required. Empirical methods of design often give unsatisfactory results and are not to be recommended.

DESIGN OF BRIDGE PIERS.—Bridge piers must be designed (1) for the total vertical load due to the dead load of the span and the live load on the span, and the weight of the pier; (2) for wind pressure on the pier and the bridge; (3) to withstand floating drift and ice; and (4) to take the longitudinal thrust due to stopping a car or train on the bridge, and due to temperature when the rollers do not move freely. The wind pressures are calculated as specified in specifications for bridges, and are assumed to act in the vertical line of the center of the pier; on the top chord of the truss; the bottom chord of the truss; 6 or 7 feet above the base of the rail; and at the center of gravity of the exposed part of the pier. The total wind moment is then calculated about the leeward edge of the base of the pier, and the maximum stresses on the foundation due to direct load and wind are calculated in the same manner as the calculation of the pressures of abutments.

The effect of the current of the stream and of floating ice and drift are difficult to calculate. The pressure of a flowing stream on an obstruction is given by the formula

$$P = m \cdot w \cdot a \cdot \frac{V^2}{2g}$$

where P = the total pressure on the surface; m = a constant; w = weight of a cubic foot of water; a = area of wetted surface normal to the current in square feet; v = velocity of current in feet per second; and g = acceleration due to gravity = 32.2 feet. The value of m varies with the shape and the dimensions of the pier. Weisbach's Mechanics gives the following data:—For a prism three times as long as broad, m = 1.33. For a pier five or six times as long as broad and with a cutwater having plane faces and an angle of 30 degrees between the cutwater faces, m = 0.48. For a square pier, m = 1.28, and for a circular pier, m = 0.64.

The maximum pressure due to floating ice will be the crushing strength of the ice, which varies from 400 to 800 lb. per sq. in. The principal danger from floating ice and drift is that the current of the stream will be deflected downward and will gouge out the material around and under the pier and cause failure. To prevent this it is quite common to build piers with a "break-water," "starkwater," "cutwater," or nose that will deflect drift and ice, or to put in a pile protection on the upstream side of the pier. If the water can get under the pier the buoyancy of the water must be considered in calculating the stability of the pier. If there is danger of scouring then it is well to deposit large stones and riprap around the base of the pier.

Batter.—Piers and abutments are seldom battered more than one inch to one foot of vertical height, or less than one-half inch to the foot, although high piers are sometimes battered only one-fourth inch to one foot.

ALLOWABLE PRESSURES ON FOUNDATIONS.—The allowable pressures on foundations depend upon the material, the drainage, the amount of lateral support given by the adjacent material, the depth of the foundation, and other conditions, so that it is not possible to give data that will be more than an aid to the judgment. If properly designed a moderate settlement of some particular structure may do no harm, while a less settlement in another structure may be disastrous. Professor I. O. Baker gives the values in Table I in his "Masonry Construction."

TABLE I.
SAFE BEARING POWER OF SOILS.*

Kind of Material.	Safe Bearing Power in Tons per Square Foot.	
	Min.	Max.
Rock hardest in thick layers in bed	200	—
Rock equal to best ashlar masonry	25	30
Rock equal to best brick	15	20
Rock equal to poor brick	5	10
Clay in thick beds, always dry	4	6
Clay in thick beds, moderately dry	2	4
Clay soft	1	2
Gravel and coarse sand, well cemented	8	10
Sand compact and well cemented	4	6
Sand clean, dry	2	4
Quicksand, alluvial soils, etc.	0.5	1

Present practice is more nearly given by the values in Table II. Foundations should never be placed directly on quicksand.

TABLE II.
ALLOWABLE BEARING ON FOUNDATIONS.

Kind of Material.	Tons per Square Foot.
Soft clay or loam	1
Ordinary clay and dry sand mixed with clay	2
Dry sand and dry clay	3
Hard clay and firm, coarse sand	4
Firm, coarse sand and gravel	6
Shale rock	8
Hard rock	20

* Baker's "Masonry Construction," John Wiley & Sons.

Mr. E. L. Corthell gives the summary of the pressures on deep foundations in Table III.

TABLE III.

ACTUAL PRESSURES ON DEEP FOUNDATIONS.*

Material.	Number of Examples.	Pressure in Tons per Square Foot.		
		Maximum.	Minimum.	Average.
Actual Pressures which Showed No Settlement.				
Fine sand................	10	5.4	2.25	4.5
Coarse sand and gravel.....	33	7.75	2.4	5.1
Sand and clay.............	10	8.5	2.5	4.9
Alluvium and silt..........	7	6.2	1.5	2.9
Hard clay.................	16	8.0	2.0	5.08
Hard pan..................	5	12.0	3.0	8.7
Actual Pressures which Showed Settlement.				
Fine sand..............	3	7.0	1.8	5.2
Clay.....................	5	5.6	4.5	5.2
Alluvium and silt..........	2	7.6	1.6	...
Sand and clay.............	3	7.4	1.6	3.3

The data in Table III shows that great care must be used in determining on the allowable pressure for any particular foundation, and that safe values for the bearing power of soils should only be used as an aid to the judgment of the engineer.

WATERWAY FOR BRIDGES.—The clear waterway for bridges should be ample; great care should be used to prevent floating logs and debris from clogging up the opening. The necessary waterway depends upon the character and size of the runoff area, the slope and size of the stream and upon other local conditions. The "Dun Drainage Table," Table IV, will be of assistance in assisting the judgment of the engineer in determining on the proper waterway for any bridge.

Many formulas have been proposed for determining the waterway of culverts and bridges. The formula best known to the author is that proposed by Professor A. N. Talbot. It is

$$A = c\sqrt[4]{M^3}$$

where A = area of the required opening in sq. ft.;

M = area of drainage basin in acres;

c = a coefficient varying with the slope of the ground, slope of the drainage area, character of the soil and character of vegetation.

Professor Talbot gives the following values of c : $c = \frac{2}{3}$ to 1 for steep and rocky ground; $c = \frac{1}{3}$ for rolling agricultural country, subject to floods at times of melting snow, and with the length of valley 3 to 4 times its width; $c = \frac{1}{5}$ to $\frac{1}{6}$ for districts not affected by accumulated snow and where the length of the valley is several times its width.

PREPARING THE FOUNDATIONS.—The preparation of the site of the abutment or pier will depend upon the conditions and character of the material.

Rock.—Where the water can be excluded, the rock should be cleared of all overlying material and disintegrated rock. The surface is then leveled up either by cutting off the projections or by depositing a layer of concrete.

Hard Ground.—The material should be excavated well below the frost and scour line. Where the foundations cannot be carried low enough to prevent undermining, piles should be driven at about 2½ to 3 ft. centers over the foundation.

* "Allowable Pressures on Deep Foundations" by E. L. Corthell. John Wiley & Sons

TABLE IV.

The Dun Drainage Table.*

Atchison, Topeka & Santa Fe Railway System.

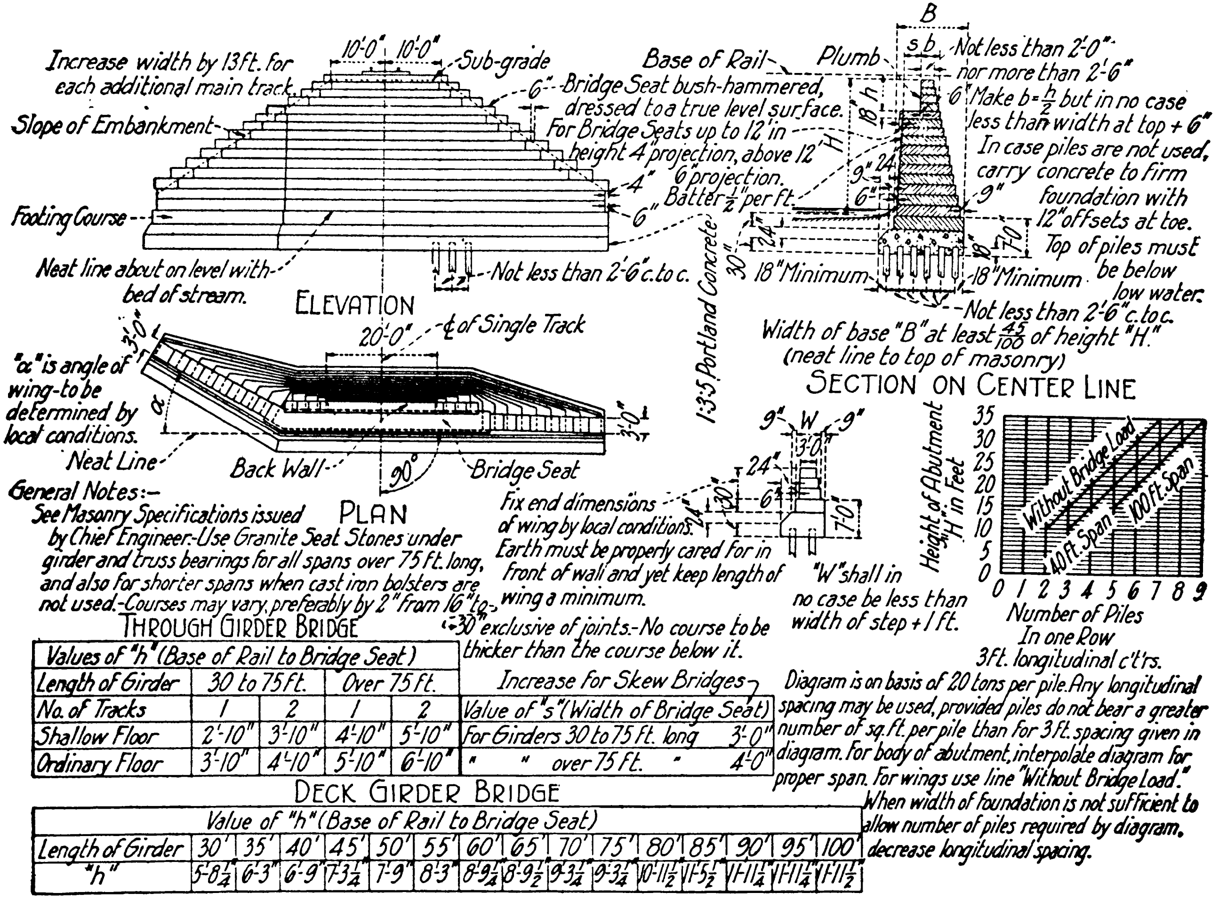
Areas Drained in Square Miles.	AREAS OF WATERWAY. Missouri and Kansas.	Cast Pipe. Banks over 15 Ft. Use 80 Per Cent.	Box and Arch Culverts. 1st Fig. = Diam. 2d Fig. = Bench.	Percentage of Column 2. Illinois.	Indian Territory.	Texas.	New Mexico.
1	2	3	4	5	6	7	8
.01	2.0	1–24 in.	2 x 1 B	East of Streator use 60 per cent.	South of Purcell use Texas Column.	Use Column 2.	Use Column 2.
.02	4.0	1–24 “	2 x 2 “				
.03	6.0	1–30 “	2 x 3 “				
.04	7.5	1–36 “	2½ x 3 “				
.05	9.0	1–42 “	3 x 3 “				
.06	10.5	1–42 “	3½ x 3 “				
.07	12.0	1–48 “	3 x 4 “				
.08	13.5	2–36 “	D 2½ x 3 “				
.09	15	2–36 “	“ 2½ x 3 “				
.10	16	2–36 “	“ 3 x 3 “				
.15	25	2–48 “	“ 3 x 4 “				
.20	32	3–42 “	6 x 4 A				
.25	38	3–48 “	6 x 5 “				
.30	44	....	6 x 5½ “				
.35	51	....	8 x 4½ “				
.40	56	....	8 x 5 “				
.45	62	....	8 x 6 “				
.50	66	....	8 x 6 “				
.55	70	....	8 x 6½ “				
.60	74	....	10 x 4½ “				
.65	78	....	10 x 5 “				
.70	81	....	10 x 5½ “				
.75	85	....	10 x 6 “				
.80	88	....	10 x 6½ “	West of Streator use 80 per cent.	North of Purcell use Column 2.		
.85	91	....	10 x 6½ “				
.90	94	....	10 x 6½ “				
.95	97	....	12 x 5 “				
1.0	100	....	12 x 5 “				98½
1.1	110	....	12 x 6 “			105	98½
1.2	120	....	12 x 7 “			105	98½
1.3	130	....	12 x 8 “			105	98½
1.4	140	....	14 x 6½ “			105	98½
1.5	150	....	14 x 7 “			105	98½
1.6	160	....	16 x 6½ “			105	98½
1.7	170	....	16 x 7 “			105	98½
1.8	180	....	16 x 7½ “			105	98½
1.9	190	....	16 x 8 “			105	98½
2.0	200	....	18 x 7 “			105	98½
2.2	220	....	18 x 8 “			105	98½
2.4	240	....	18 x 9½ “			105	98½
2.6	260	....	20 x 8 “			105	98½
2.8	280	....	20 x 9 “			105	98½
3.0	300	....	20 x 9½ “	West of Streator use 80 per cent. East of Streator use 60 per cent.	North of Purcell use Column 2. South of Purcell use Texas Column.	105	98½
3.2	321	....	22 x 8½ “			105	98½
3.4	340	....	22 x 9 “			105	98½
3.6	357	....	24 x 8½ “			105	98½
3.8	373	....	24 x 9 “			105	98½
4.0	388	....	28 x 7 “			105	97
4.2	403	....	28 x 7½ “			105	97
4.4	417	....	28 x 8 “			105	97
4.6	430	....	28 x 8½ “			105	97
4.8	443	....	28 x 9 “			105	97
5.0	455	....	28 x 9½ “			105	97
5.5	483	....	28 x 10 “			105	97
6.0	509	....	32 x 7½ “			105	97
6.5	533	....	32 x 8 “			105	97
7.0	556	....	32 x 9 “			105	97
7.5	579	....	32 x 10 “			105	97
8.0	601	....	32 x 11 “			105	97
8.5	622	....	32 x 11½ “			105	97
9.0	641	....	32 x 12 “			105	93½
9.5	660	....	32 x 12½ “			105	93½
10	679	....	32 x 13 “			105	93½

Areas Drained in Square Miles	AREAS OF WATERWAY. Missouri and Kansas.	Percentage of Column 2. Illinois.	Indian Territory.	Texas.	New Mexico.
1	2	5	6	7	8
11	710	East of Streator use 60 per cent.	South of Purcell use Texas Column.	105	93½
12	740			105	93½
13	775			105	93½
14	805			105	93½
15	835			105	93½
16	865			105	94
17	890			105	94
18	920			105	94
19	945			105	94
20	970			105	94
22	1,015			105	94
24	1,060			110	94
26	1,100			110	92
28	1,140			110	92
30	1,180			110	92
32	1,220			110	92
34	1,255			110	92
36	1,290			110	91
38	1,320			110	91
40	1,350			110	91
45	1,435			110	91
50	1,510			110	89½
55	1,580	West of Streator use 80 per cent.	North of Purcell use Column 2.	115	89½
60	1,650			115	89½
65	1,720			115	88
70	1,780			115	88
75	1,840			115	88
80	1,900			115	86½
85	1,960			115	86½
90	2,015			115	86½
95	2.065			115	86½
100	2,120			120	85
110	2,220			120	85
120	2,315			120	85
130	2,405			125	83½
140	2,500			125	83½
150	2,580			130	82
160	2,665			130	82
170	2,745			130	80½
180	2,820			130	80½
190	2,900			130	79
200	2,970			130	79
220	3,115	West of Streator use 80 per cent. East of Streator use 60 per cent.	North of Purcell use Column 2. South of Purcell use Texas Column.	130	77½
240	3,245			130	77½
260	3,370			130	76
280	3,495			130	76
300	3,615			130	74½
325	3,770			130	74½
350	3,900			130	73
375	4,035			130	73
400	4,165			130	71½
450	4,385			130	70
500	4,610			130	68½
550	4,825			130	67
600	5,030			130	65½
700	5,420			130	62½
800	5,800			130	59½
900	6,080			130	56½
1,000	6,380			130	.....
2,000	8,820			130	.....
3,000	10,640			130	.....
4,000	12,160			130	.....
5,000	13,500			130	.....

The above classification by states is for convenience only, and merely denotes the general characteristics of topography and rainfall.

Column 2 in this table is prepared from observations of streams in Southwest Missouri, Eastern Kansas, Western Arkansas and the southeastern portions of the Indian Territory. In all this region steep, rocky slopes prevail and the soil absorbs but a small percentage of the rainfalls. It indicates larger waterways than are required in Western Kansas and level portions of Missouri, Colorado, New Mexico and Western Texas.

* American Railway Engineering Association, Vol. 12, p. 484. This report also contains an elaborate report on Runoff and Waterways for Culverts.



THROUGH GIRDER BRIDGE

Values of "h" (Base of Rail to Bridge Seat)				
Length of Girder	30 to 75 ft.		Over 75 ft.	
No. of Tracks	1	2	1	2
Shallow Floor	2'-10"	3'-10"	4'-10"	5'-10"
Ordinary Floor	3'-10"	4'-10"	5'-10"	6'-10"

Increase for Skew Bridges

Value of "s" (Width of Bridge Seat)	
For Girders 30 to 75 ft. long	3'-0"
" " over 75 ft. "	4'-0"

DECK GIRDER BRIDGE

Value of "h" (Base of Rail to Bridge Seat)															
Length of Girder	30'	35'	40'	45'	50'	55'	60'	65'	70'	75'	80'	85'	90'	95'	100'
"h"	5'-8¼"	6'-3"	6'-9"	7'-3¼"	7'-9"	8'-3"	8'-9¼"	8'-9½"	9'-3¼"	9'-3¼"	10'-11½"	11'-5½"	11'-11¼"	11'-11¼"	11'-11½"

FIG. 3. MASONRY ABUTMENTS, BALTIMORE AND OHIO RAILWAY.

Soft Ground.—The materials should be excavated to a solid stratum or piles spaced about 2½ to 3 ft. centers should be driven over the foundation to a good refusal. The piles should be cut off below low water level to carry a timber grillage, or concrete may be deposited around the heads of the piles. Where water cannot be excluded it will be necessary to use one of the following methods: open caisson, crib, coffer dam, or pneumatic caisson.

In using an open caisson the masonry is built up or the concrete is deposited in a water tight box built of heavy timbers or of reinforced concrete, the caisson being sunk as the pier is built up.

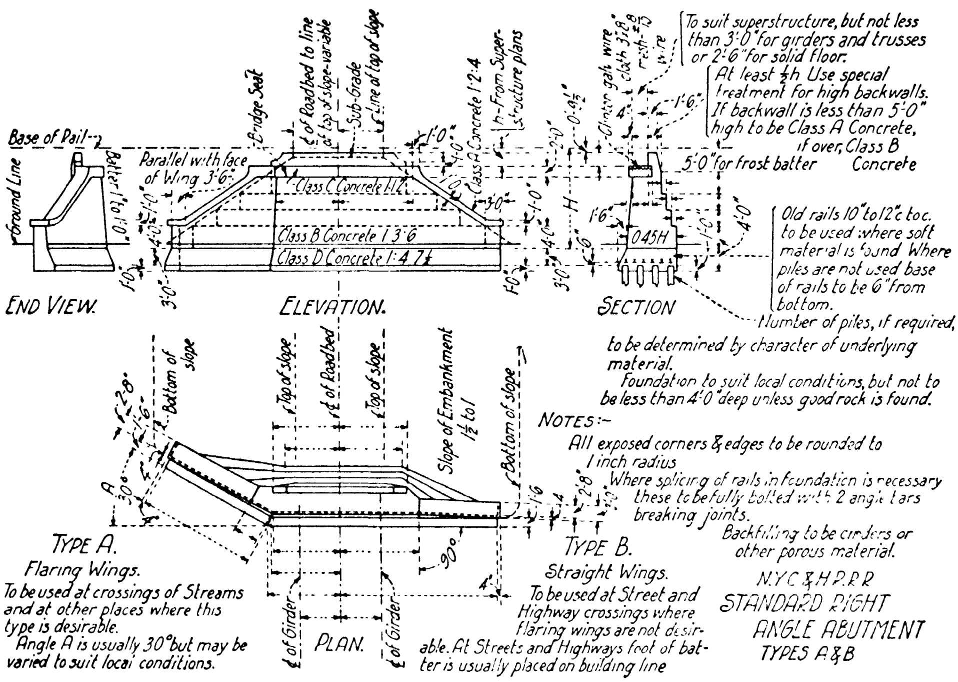


FIG. 4. MASONRY ABUTMENTS, N. Y. C. & H. R. R. R.

The caisson is commonly floated into place and then is sunk on piles which have been sawed off to receive it, or on a solid rock foundation. The sides of timber caissons are usually removed after the pier is completed.

Timber cribs are made of squared timbers placed transversely and longitudinally, and bolted together so as to form a solid structure with open pockets. The crib is sunk by loading the pockets with stone. No timber should be left above the low water mark in open caissons or cribs.

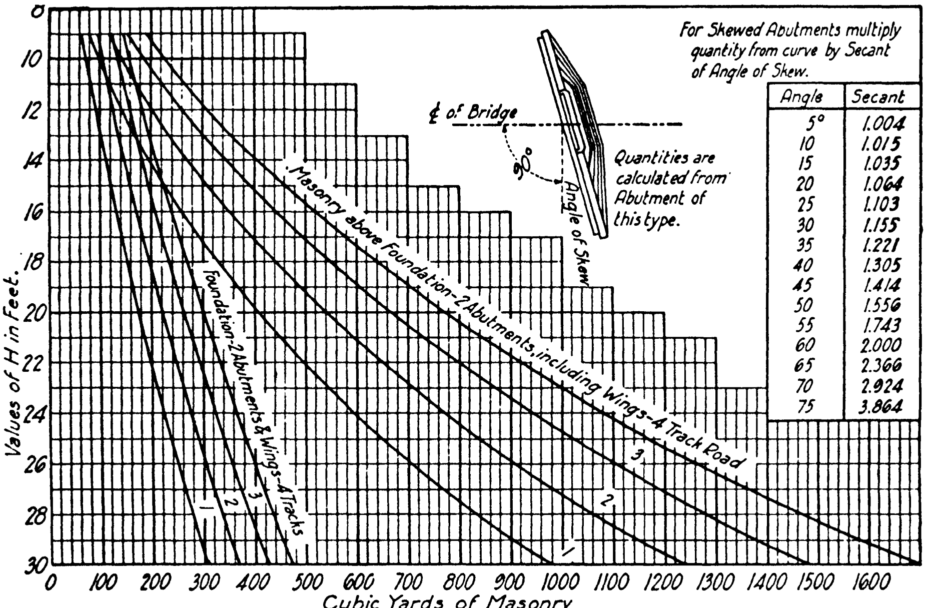
A coffer dam is usually made by driving two rows of sheet piling around the pier, the space between the rows of piling being filled with clay puddle. For small depths a single row of sheet piling is often sufficient. Where the depth is too great for one length of sheet piling, additional rows are driven inside the first. Steel sheet piling is now much used for difficult foundations. Steel sheet piling can be driven through ordinary drift and similar material, is not limited in depth, and is practically water tight when used in a single row. It can be drawn and used again. It is almost impossible to shut off all the water with a coffer dam, and pumps should be provided.

Pneumatic caissons should only be used under the direction of experienced engineers and will not be considered here.

For details of sinking piers see Jacoby & Davis' "Foundations of Bridges and Buildings", McGraw-Hill Book Company.

EXAMPLES OF RAILWAY BRIDGE ABUTMENTS.—Standard stone masonry abutments designed by the Baltimore & Ohio Railway are shown in Fig. 3. These abutments are to be used for deck and through girder spans. The plans are worked out in detail and give data for different conditions.

Standard designs for a straight abutment and for a wing abutment designed by the N. Y. C. & H. R. R. R. are shown in Fig. 4. Data for different conditions are given on the plans. The quantity of masonry and of old railroad rails required for the N. Y. C. & H. R. R. R. abutments shown in Fig. 4 are given in Fig. 5. The wings are the length required for a flare of 30 degrees and a side slope of roadway of 1½ to 1.



Old Rails in Foundation - 65# Rails spaced 10" to 12" c. to c.
Weight in Tons - 2 Abutments - Straight Wings.

H	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1 Track	6.3	7.0	7.8	8.4	9.2	10.0	10.8	11.5	12.4	13.2	14.0	15.0	16.0	17.0	18.0	19.0	20.0	21.0	22.0	23.2	24.4	25.6
2 Tracks	9.0	9.8	10.6	11.4	12.2	13.2	14.0	15.0	16.0	17.0	18.0	19.0	20.0	21.0	22.0	23.2	24.4	25.6	26.8	28.0	29.2	30.5
3 "	11.4	12.2	13.2	14.2	15.0	16.2	17.2	18.2	19.2	20.2	21.4	22.5	23.6	24.8	26.0	27.2	28.4	29.6	31.0	32.9	33.6	35.0
4 "	13.5	14.5	15.5	16.5	17.5	18.6	19.6	20.8	22.0	23.1	24.2	25.5	26.8	28.0	29.4	30.6	32.0	33.4	35.0	36.2	37.6	39.0

NOTE:—H equals distance from top of Foundation to Base of Rail.
Quantities shown by curves are NET.
Foundation based on depth of 4 feet.

N.Y.C. & H.R.R.R.
QUANTITIES IN STANDARD ABUTMENTS
TYPES A & B

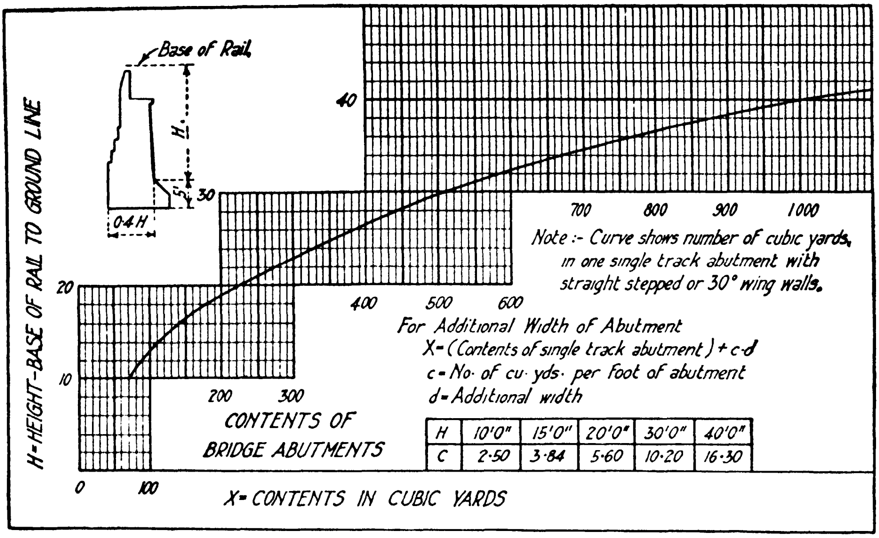
FIG. 5. QUANTITIES IN MASONRY ABUTMENTS, N. Y. C. & H. R. R. R.

The quantity of concrete in single track railway bridge abutments as designed by the Illinois Central R. R. are given in Fig. 6. The quantities in double track abutments may be calculated as shown in Fig. 6.

Cooper's Standard Abutments—The abutment in (*a*), Fig. 7, is from Cooper's "General Specifications for Foundations and Substructures of Highway and Electric Railway Bridges." The length, *l*, and the thickness, *a*, for highway and single track electric railway bridges are as

given, and are proportional for intermediate spans. These abutments may be made of either first-class stone masonry, or first-class Portland cement concrete.

For double track electric railway bridges add one foot to the value of a in Fig. 7. The minimum thickness of the wall at any point is to be 0.4 of the height. The length of the wing walls will be determined by local conditions.



H	10'0"	15'0"	20'0"	30'0"	40'0"
C	2.50	3.84	5.60	10.20	16.30

FIG. 6. QUANTITIES IN MASONRY ABUTMENTS, ILLINOIS CENTRAL RAILROAD.

The abutment without wing walls in (*b*), Fig. 7. has the same dimensions as the abutment with wing walls. The width for single track electric railways may be taken as 14 ft., double track 26 ft. The approximate cubical quantities in abutments without wing walls are given in Fig. 7

RAILWAY BRIDGE PIERS.—Standard piers for railway bridges as designed by the N. Y. C. & H. R. R. R. are shown in Fig. 8. Dimensions and data for different spans and heights of piers are given on the plans. The quantities of masonry in the standard plans shown in Fig. 8 are given in Fig. 9, for deck spans and for through spans.

Quantities of masonry in piers for deck plate girder spans are given in Fig. 10 and for through girder and truss spans in Fig. 11. These piers were designed and the estimates were prepared by the bridge department of the Illinois Central Railroad.

Illinois Central Railroad Pier.—Details of a concrete pier designed and built by the Illinois Central Railroad are shown in Fig. 12. The pier rests on timber piles spaced as shown. The "starkwater" is reinforced with an 8 in. I beam.

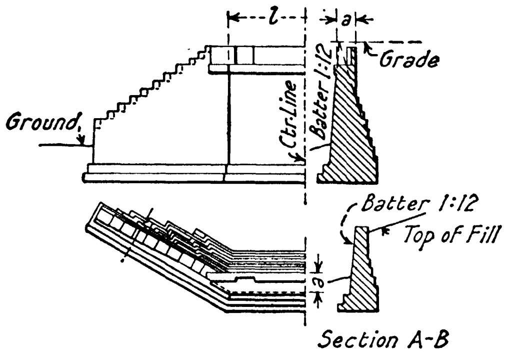
Cooper's Standard Masonry Piers.—The masonry pier in Fig. 13 is from Cooper's "General Specifications for Substructures of Highway and Electric Railway Bridges." The length, l, and the thickness, a, for highway and single track electric railway bridges are given in Fig. 13. These piers may be made of either first-class stone masonry, or first-class Portland cement concrete.

For double track electric railway bridges add one foot to l, and 6 inches to a. The width, w = center to center of trusses, and may ordinarily be taken 14 ft. for single track, and 26 ft. for double track through bridges. Where drift and logs are liable to injure the pier the nose of the cut-water should be protected with a steel angle or plate. The approximate cubical contents of the piers are given in Fig. 13.

STEEL TUBULAR PIERS.—Steel tubular piers are made of steel plates riveted together and filled with concrete. Where the piers are founded on soft material, piles are driven in the

bottom of the tube, the piles being sawed off below the water line. The piles should extend at least two diameters of the tube above the bottom. The tubes are braced transversely by means of struts and tension diagonals above high water and by diaphragm bracing below high water. Where the piers will be subject to blows from floating drift or logs they should be protected by a timber cribwork or other device.

Cooper's Standards.—The tubular piers in Fig. 14 are from Cooper's "General Specifications for Foundations and Substructures for Highway and Electric Railway Bridges." Cooper specifies

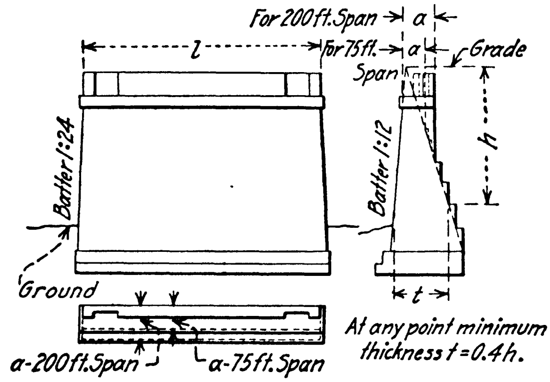


(a) HIGHWAY ABUTMENT WITH WING WALLS

DIMENSIONS OF MASONRY ABUTMENTS WITH WING WALLS

Distance, a	Span, Feet	Length, l.
2′ 6″	50	w + 4′ 0″
2′ 8″	100	w + 5′ 0″
3′ 0″	150	w + 5′ 9″
3′ 4″	200	w + 6′ 6″
3′ 6″	250	w + 7′ 0″

w = center to center of trusses, 14 ft. for single track, 26 ft. for double track.



(b) HIGHWAY ABUTMENT WITHOUT WING WALLS

APPROXIMATE QUANTITIES IN CU. YDS. OF ONE MASONRY ABUTMENT WITHOUT WING WALLS

Span Feet	Roadway	Depth Footing Below Grade				
		10′	15′	20′	25′	30′
100	12 Feet	20	39	67	100	145
	20 Feet	28	56	95	145	206
	E, Single T.	21	44	75	112	160
	E, Double T.	36	72	120	183	260
300	12 Feet	22	45	77	116	165
	20 Feet	31	63	106	161	227
	E, Single T.	25	50	85	128	181
	E, Double T.	49	84	141	210	296

FIG. 7. MASONRY ABUTMENTS FOR ELECTRIC RAILWAY AND HIGHWAY BRIDGES. COOPER'S STANDARDS.

a minimum thickness of $\frac{3}{8}$ in. for plates below and $\frac{1}{4}$ in. above the high water. The minimum size of tubular piers are as given in Fig. 14.

A steel tubular pier with a timber crib protection is given in Fig. 14. The crib is filled with loose rock.

A steel oblong pier, as designed by Cooper, is given in Fig. 15. The center of the truss is to come $a/2$ + one ft. from the end of the pier. The width a, as specified by Cooper, is given in Fig. 15.

American Bridge Company Standards.—The American Bridge Company's standard tubular piers are shown in Fig. 16. The minimum diameters for a height of 15 feet to carry a single span,

and data on piers, pier beams and pier bracing are given in Fig. 16. In calculating the weight of a pier add one foot to the length of each tube. The weight of the concrete in two tubes is given in Fig. 16. The concrete is assumed to fill the tube, and the space occupied by piles should be deducted. The number of piles required for different diameters of tubes is given. The number of piles required for large tubes agrees quite closely with Cooper's Specifications, but the number for small tubes is very much less.

Pier Beams.—The sizes of pier beams required for different panel lengths and clear distance between tubes in feet are given in Fig. 16. The pier beam should be assumed as one foot longer than the clear distance between the tubes, in calculating the weight of the beams.

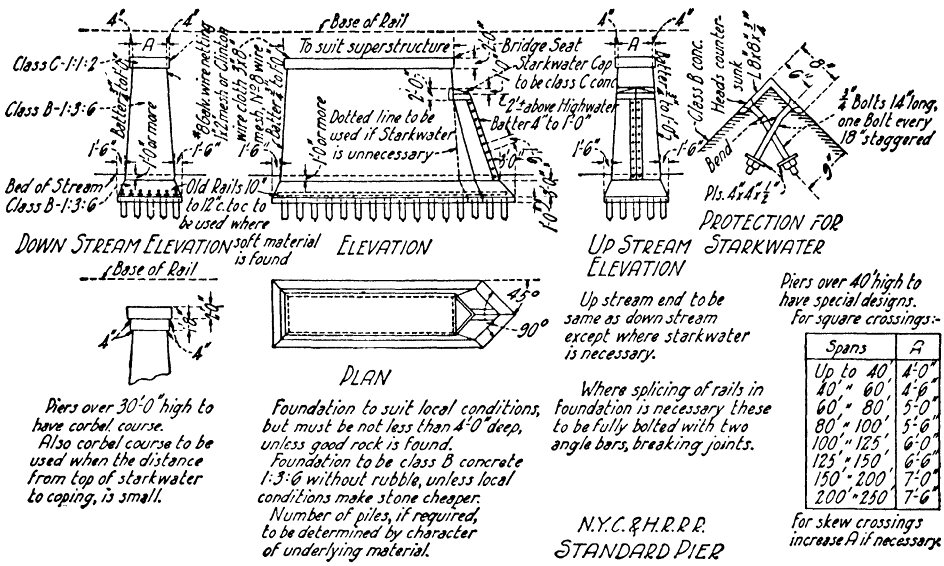


FIG. 8. MASONRY PIERS, N. Y. C. & H. R. R. R.

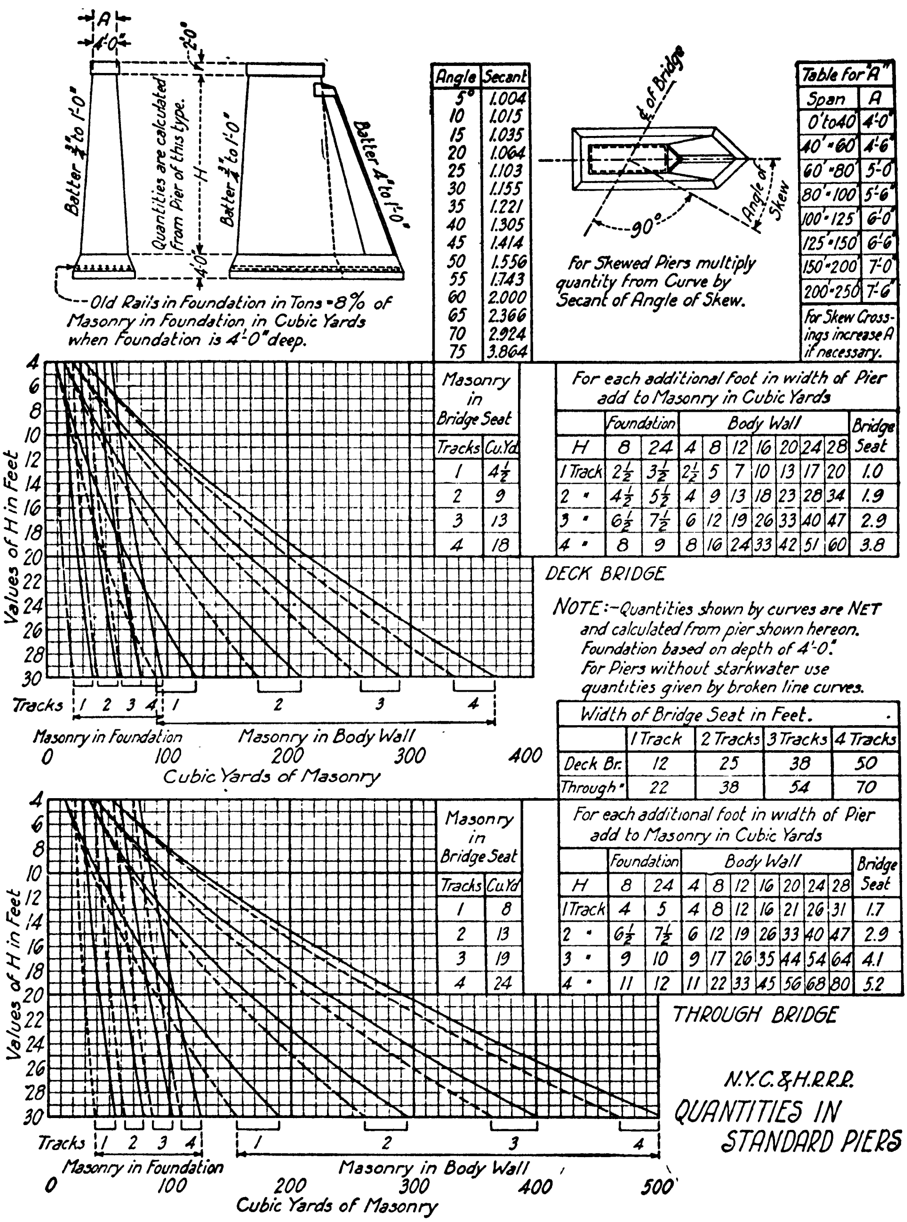
Pier Bracing.—The pier bracing for piers supporting the ends of two spans are given in Fig. 16. If the spans are unequal in length, enter the table with one-half of the algebraic sum of the spans. For example, for a pier carrying a 75 ft. and a 125 ft. span, enter the diagram with a span of 100 ft. Steel tubular piers should never be used for end abutments carrying a fill.

In calculating the weight of the diagonal bars the length of the bar should be multiplied by the weight per foot as obtained from a handbook, and the details for one bar added to the product. In calculating the weight of the struts add one foot to the clear length.

Pier Caps.—Tubular piers may be capped with steel plate caps, may be finished with concrete, or may have a stone pedestal block. The weights given in Fig. 16 do not include the weights of steel caps.

Specifications for Steel Tubular Piers for Highway and Electric Railway Bridges.—The plates for the tubes shall be not less than $\frac{1}{4}$ in. thick for tubes up to 30 in. in diameter, not less than $\frac{5}{16}$ in. for tubes from 30 to 48 in. in diameter, and not less than $\frac{3}{8}$ in. for tubes from 48 to 72 in. in diameter. Where the plates are in contact with the soil the thickness shall be increased at least $\frac{1}{16}$ in. For $\frac{5}{16}$ in. plate and less use $\frac{5}{8}$ in. rivets; for $\frac{3}{8}$ in. plate and over use $\frac{3}{4}$ in. rivets.

The horizontal seams shall be single lap joints riveted with a pitch of 4 diameters of rivet, while the vertical seams shall preferably be butt riveted with single riveting spaced 4 diameters of rivet, up to 48 in. diameter of tubes, and double riveting with 3 in. spacing for tubes of larger diameter.



Angle	Secant
5°	1.004
10	1.015
15	1.035
20	1.064
25	1.103
30	1.155
35	1.221
40	1.305
45	1.414
50	1.556
55	1.743
60	2.000
65	2.366
70	2.924
75	3.864

Table for "A"	
Span	A
0' to 40'	4'-0"
40' " 60'	4'-6"
60' " 80'	5'-0"
80' " 100'	5'-6"
100' " 125'	6'-0"
125' " 150'	6'-6"
150' " 200'	7'-0"
200' " 250'	7'-6"
For Skew Crossings increase A if necessary.	

Deck Bridge

Masonry in Bridge Seat	
Tracks	Cu. Yd.
1	$4\frac{1}{2}$
2	9
3	13
4	18

For each additional foot in width of Pier add to Masonry in Cubic Yards

	Foundation		Body Wall							Bridge Seat
H	8	24	4	8	12	16	20	24	28	
1 Track	$2\frac{1}{2}$	$3\frac{1}{2}$	$2\frac{1}{2}$	5	7	10	13	17	20	1.0
2 "	$4\frac{1}{2}$	$5\frac{1}{2}$	4	9	13	18	23	28	34	1.9
3 "	$6\frac{1}{2}$	$7\frac{1}{2}$	6	12	19	26	33	40	47	2.9
4 "	8	9	8	16	24	33	42	51	60	3.8

Width of Bridge Seat in Feet.				
	1 Track	2 Tracks	3 Tracks	4 Tracks
Deck Br.	12	25	38	50
Through "	22	38	54	70

Through Bridge

Masonry in Bridge Seat	
Tracks	Cu. Yd.
1	8
2	13
3	19
4	24

For each additional foot in width of Pier add to Masonry in Cubic Yards

	Foundation		Body Wall							Bridge Seat
H	8	24	4	8	12	16	20	24	28	
1 Track	4	5	4	8	12	16	21	26	31	1.7
2 "	$6\frac{1}{2}$	$7\frac{1}{2}$	6	12	19	26	33	40	47	2.9
3 "	9	10	9	17	26	35	44	54	64	4.1
4 "	11	12	11	22	33	45	56	68	80	5.2

FIG. 9. QUANTITIES IN MASONRY PIERS, N. Y. C. & H. R. R. R.

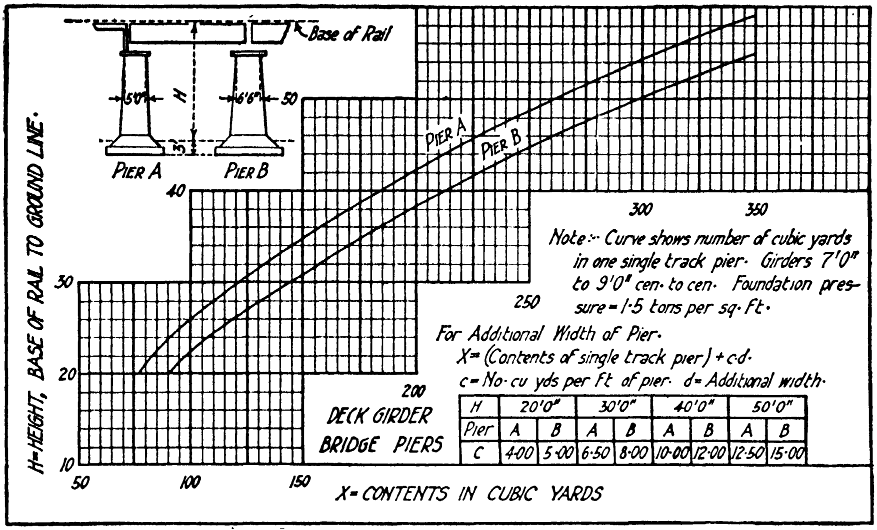
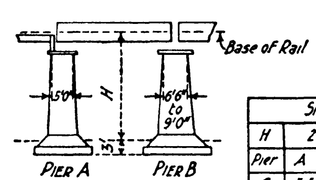


FIG. 10. QUANTITIES IN MASONRY PIERS FOR DECK GIRDERS, ILLINOIS CENTRAL RAILROAD.



X = Number of cubic yards in one single track pier.
Girders or trusses—15'0" to 18'0" c. to c. Footing Pressure = 1·5 tons/sq. ft.
For Additional Width of Pier :—
X = (Contents of Single Track Pier) + c·d
c = No. of cu. yds. per ft. of pier. d = Additional width.

	SPANS 150'0" OR UNDER								SPANS OVER 150'0"							
H	20'0		30'0"		40'0"		50'0"		20'0"		30'0"		40'0		50'0"	
Pier	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
C	5·5	6·0	8·0	9·0	12·0	13·5	14·5	16·5	7·1	8·2	10·0	12·0	13·0	16·0	17·0	21·0

VALUES OF X FOR SPANS 30'0" TO 400'0" IN LENGTH

H	30'0"		40'0"		50'0"		60'0"		70'0"		80'0"		90'0"		100'0"	
	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
20'0"	121	143	121	143	127	149	127	149	130	152	138	160	147	169	158	180
30'0"	208	244	208	244	219	255	219	255	221	257	224	260	227	263	234	270
40'0"	314	364	314	364	320	370	320	370	322	372	325	375	336	386	340	390
50'0"	410	470	410	470	415	477	415	477	417	477	420	480	422	482	424	484

H	125'0"		150'0"		175'0"		200'0"		250'0		300'0"		350'0"		400'0"	
	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
20'0"	160	198	184	228	251	280	275	324	261	306	270	371	285	351	298	370
30'0"	252	310	306	366	320	384	328	400	326	404	340	425	370	467	399	516
40'0"	342	430	395	482	420	520	448	550	435	545	453	570	480	632	513	673
50'0"	485	590	501	601	548	648	576	703	550	710	576	726	608	793	645	852

Note:—Piers for all spans, 200'0" or more in length are designed for Pile Foundations

CONTENTS OF SINGLE TRACK, THROUGH SPAN PIERS

FIG. 11. QUANTITIES IN MASONRY PIERS FOR THROUGH SPANS, ILLINOIS CENTRAL RAILROAD.

The bracing of piers shall be designed to take all the wind forces specified to come on the bridge. Diaphragm webs are to be used up to well above high water for piers located in the stream or where floating materials may find lodgment. Oblong piers shall be braced against inside and outside pressure. Piers exposed to injury from floating logs and drift shall be protected.

The tubes should be painted inside and out with two coats of red lead and linseed oil, or other prescribed paint.

The materials and workmanship shall comply with the specifications for the highway bridge superstructure.

Erection.—Where the bottom will permit, the tubes shall be sunk well below possible scour by loading the tube and excavating the material from the inside. For this purpose a clamshell bucket is very effective. Driving the tube with a pile driver will cut off the rivets in the horizontal seams and will not be permitted. After the tube is sunk, piles are to be driven inside of the steel shell, as closely together as possible, using care to get no pile nearer than 4 to 6 in. to the steel shell. The piles shall be driven to a good refusal, and the tops sawed off below the low water mark and reaching at least 2 diameters of the tube above the bottom. The space inside the tubes shall then be filled with concrete well tamped. Concrete should not be deposited in running water if possible to prevent it.

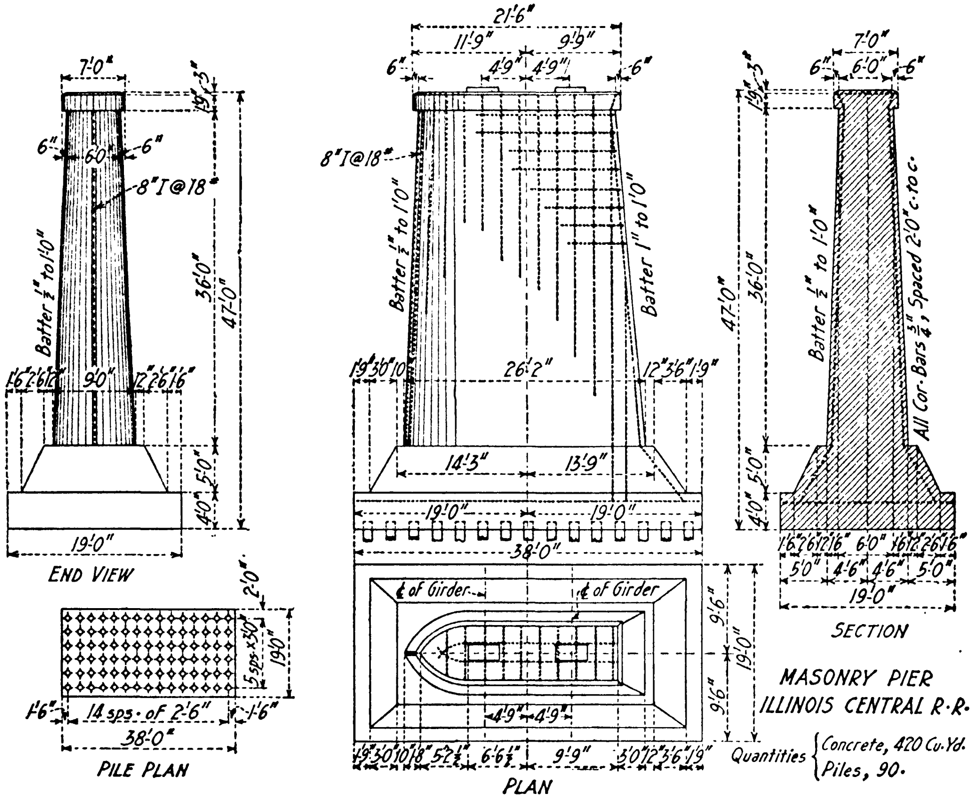
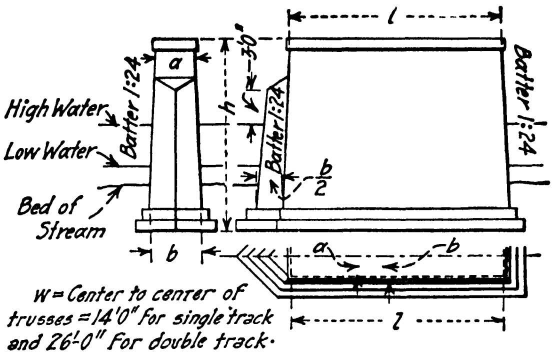


FIG. 12. DETAILS OF ILLINOIS CENTRAL RAILROAD PIER.

Where piers are founded on rock, the tubes are to be anchored to the rock and then filled with concrete. Or cribs may be sunk on the rock and the tube set in a pocket in the crib and resting on the rock. The space outside the tube is then filled with concrete and the tube is filled with concrete in the usual manner.

Cylinder Piers for Highway Bridge, Trail, B. C.*—Steel cylinder piers were used for a steel highway bridge designed by Waddell and Harrington, Consulting Engineers, and built across the Columbia River at Trail, B. C. The main spans are 172 ft. 8 in long and are carried on piers made of two steel cylinders filled with concrete. The steel cylinders are 9 ft. in diameter at the bottom and 6 ft. in diameter at the top, and are 86 ft. long. The cylinders are made of

* Engineering News, Dec. 5, 1912.



HIGHWAY PIERS

DIMENSIONS FOR MASONRY PIER FOR HIGHWAY AND SINGLE TRACK ELECTRIC RAILWAY BRIDGES

Distance a	Span Feet	Length l
2'8"	50	w+4'0"
2'10"	75	w+4'6"
3'2"	100	w+5'0"
3'8"	150	w+5'9"
4'4"	200	w+6'6"
4'10"	250	w+7'0"
5'4"	300	w+7'6"

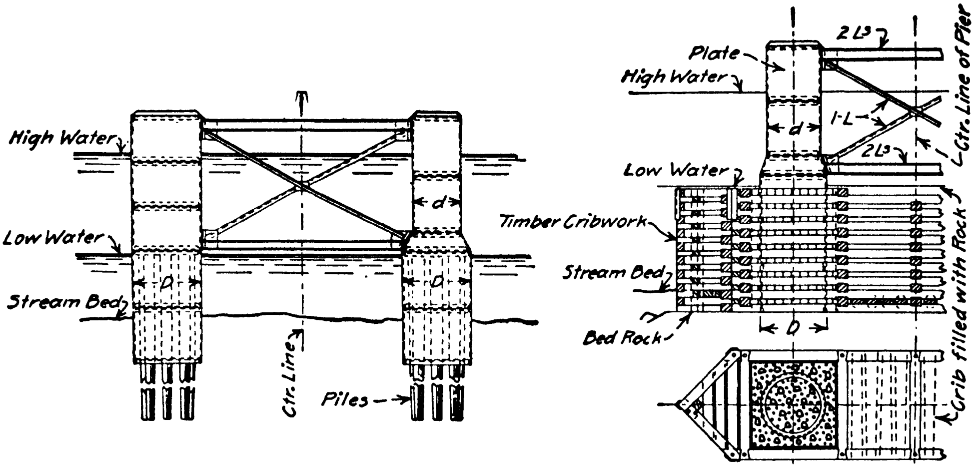
For double track Electric Railway bridges add 12" to l, and 6" to a.

APPROXIMATE CONTENTS IN CUBIC YARDS OF ONE MASONRY PIER

Spans. Feet	Roadway	Depth of Pier from Top of Coping to Bottom of Footing in Feet.				
		10	15	20	25	30
100	12 Feet	29	44	60	77	94
	20 Feet	38	59	82	108	136
	E, Single T.	31	46	62	80	100
	E, Double T.	50	75	102	132	166
150	12 Feet	34	51	70	90	111
	20 Feet	46	70	95	125	157
	E, Single T.	37	54	74	96	120
	E, Double T.	58	86	118	153	191
200	12 Feet	39	58	80	103	128
	20 Feet	53	80	109	143	178
	E, Single T.	43	63	86	112	140
	E, Double T.	66	99	135	174	217
250	12 Feet	44	66	90	116	145
	20 Feet	61	91	123	160	199
	E, Single T.	48	74	98	127	159
	E, Double T.	73	109	149	192	238
300	12 Feet	49	73	100	130	162
	20 Feet	68	101	137	177	220
	E, Single T.	54	80	109	142	178
	E, Double T.	80	120	164	210	260

FIG. 13. MASONRY PIERS FOR ELECTRIC RAILWAY AND HIGHWAY BRIDGES. COOPER'S STANDARDS.

plates ½ in. thick and are connected by a double plate web diaphragm, each diaphragm made of $\frac{5}{16}$ in. plates spaced 24 in. apart and 25 ft. high, and reaching from below low water to above high water. The diaphragms were covered and filled with concrete. The cylinders are spaced 21 ft. centers. The piers were sunk by the pneumatic process.



(a) STEEL TUBULAR PIERS

(b) CRIB CONSTRUCTION FOR STEEL TUBULAR PIERS

MINIMUM SIZES OF STEEL TUBULAR PIERS, COOPER'S STANDARDS

Span in Feet	Highway & Single Track Electric Railway			Double Track Electric Railway		
	Minimum Top, d	Diameter Bot. D.	Number of Piles	Minimum Top d	Diameter Bot D	Number of Piles
50	2'10"	3'4"	4	3'4"	4'4"	8
75	3'4"	3'9"	5	3'10"	5'6"	10
100	3'8"	4'2"	6	4'6"	6'0"	10
125	4'0"	4'7"	8	4'10"	6'4"	12
150	4'4"	5'0"	9	5'2"	7'0"	12
175	4'8"	5'6"	10	5'6"	7'6"	15
200	5'0"	5'10"	11	5'10"	8'0"	15
250	5'6"	6'4"	12	6'4"	9'0"	19

FIG. 14. STEEL TUBULAR PIERS FOR ELECTRIC RAILWAY AND HIGHWAY BRIDGES. COOPER'S STANDARDS.

STEEL CYLINDER PIERS FOR RAILWAY BRIDGES.—Steel cylinder piers have been used for the foundations of several important bridges, Table V, by the Chicago and Northwestern Railway. Mr. W. H Finley, Asst. Chief Engineer, gives the following advantages of steel cylinder piers over masonry piers.*

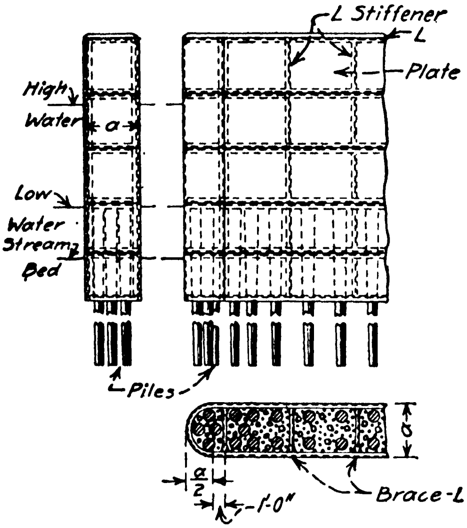
(1) "Where it is desired to provide for future second track, cylinder foundations will cost very little more for double track than for single track.

* Engineering News, Oct. 24, 1912.

(2) "Cylinder piers can be constructed under traffic with less trouble than any other type.

(3) "Cylinder piers permit of rapid sinking by open dredging where the material is favorable and sunken logs are not liable to be encountered. Air pressure can be applied readily and cheaply if it becomes necessary."

Details of the cylinder piers for the Oxford Mill Pond bridge are shown in Fig. 17, and details of the steel shells for the base of the piers are shown in Fig. 18. The bridge is 481 feet long and consists of 30 ft. and 60 ft. spans resting on piers made of two steel cylinders and a steel shell for the base, filled with concrete.



OBLONG STEEL PIERS

MINIMUM SIZES OF STEEL OBLONG PIERS
COOPER'S STANDARDS

Span in Feet	Width a: Highway and Single Track Electric Railway	Width a: Double Track Electric Railway
50	2'10"	3'4"
75	3'4"	4'0"
100	3'8"	4'6"
125	4'0"	4'10"
150	4'4"	5'2"
175	4'8"	5'6"
200	5'0"	5'10"
250	5'6"	6'4"

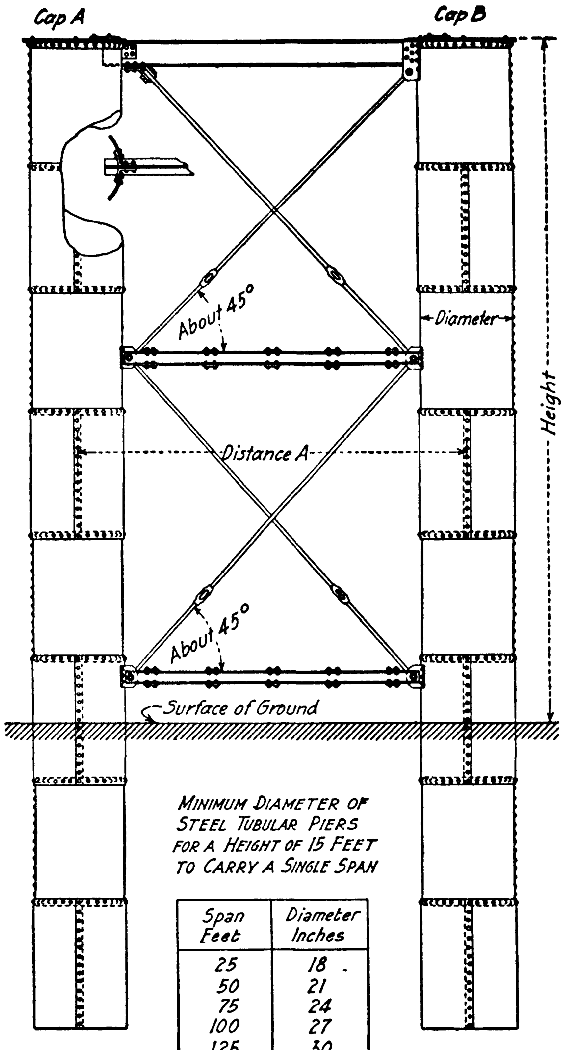
FIG. 15. STEEL OBLONG PIERS FOR ELECTRIC RAILWAY AND HIGHWAY BRIDGES. COOPER'S STANDARDS.

TABLE V.

DATA ON SEVERAL STEEL CYLINDER PIERS USED BY THE CHICAGO AND NORTHWESTERN RAILWAY.

Bridge.	Span, Ft.	Number of Cylinders in Pier.	Steel Cylinder Piers.					Steel Caisson Piers.				
			Diameter of Piers. Top, Ft.	Diameter of Piers. Bottom, Ft	Thickness of Metal, In.	Height of Pier, Ft.	No. of Piles in One Cylinder.	Width, Ft.	Length, Ft.	Thickness of Metal, In.	Height of Caisson, Ft.	No. of Piles.
Boone Viaduct	300	4 (Tower)	10	10	$\frac{5}{8}$	70	*	...	...	...	...	...
Lake Butte Des Morts Viaduct	46, 46	3	$5\frac{1}{2}$	8	$\frac{5}{16}$	34	†	...	...	...	...	...
Buffalo Lake Viaduct	30, 60	2	...	8	$\frac{3}{8}$	30	...	...	...	...	...	...
Oxford Mill Pond Viaduct	30, 60	2	6	...	$\frac{5}{16}$	34	...	10	$29\frac{1}{2}$	$\frac{3}{8}$	$19\frac{1}{2}$	49
Pekin Bridge	150	2	...	12	$\frac{3}{8}$	92	‡	...	...	...	...	...
Pekin Bridge	175	2	...	15	$\frac{3}{8}$	97	‡	...	...	...	...	...
Pekin Bridge	70	2	...	$13\frac{1}{2}$	$\frac{3}{8}$	43	30	...	...	...	...	...

* Rests on Sandstone. † Hard Clay. ‡ Rests on Hard Shale.



MINIMUM DIAMETER OF STEEL TUBULAR PIERS FOR A HEIGHT OF 15 FEET TO CARRY A SINGLE SPAN

Span Feet	Diameter Inches
25	18
50	21
75	24
100	27
125	30
150	33
175	36
200	42

Increase diameter 3″ for each additional 5 feet in height.

STEEL TUBULAR PIERS
AMERICAN BRIDGE COMPANY STANDARDS

CYLINDER PIERS

All quantities for One Pair of Tubes								Piles
Diam. of Tube	Weight per Vert. Ft. of 2 Tubes						Cu. Yd. per Vert. Ft.	No. for One Tube
	3/16″	1/4″	5/16″	3/8″	7/16″	1/2″		
15″	75#	97#	119#	142#	164#	187#	0.091	1
18	88	114	140	167	194	220	0.131	1
21	102	131	162	194	223	253	0.178	1
24	117	150	185	221	255	290	0.232	1
27	130	167	206	247	284	324	0.296	1
30	143	185	227	271	315	357	0.364	1
33	157	203	250	300	347	393	0.440	1
36	172	222	273	326	377	429	0.524	2
39	185	240	293	352	408	463	0.614	2
42	200	257	316	378	437	497	0.712	3
45	213	275	339	405	469	532	0.820	3
48	227	293	362	412	500	568	0.930	4
54		329	405	485	563	636	1.178	5
60		365	449	539	621	705	1.454	6
66			495	593	684	780	1.758	7
72			538	643	743	845	2.094	8
78				698	805	917	2.458	10
84				749	866	984	2.850	13

PIER BEAMS
FOR VARIOUS PANEL LENGTHS
AND CLEARANCES BETWEEN BEAMS.

Span Length	Clearances for Various Sizes of I Beams							
	8″ I 18#	9″ I 21#	10″ I 25#	12″ I 31½#	12″ I 40#	15″ I 42#	15″ I 50#	18″ I 55#
12′0″	9′6″	10′9″	12′3″	15′0″	16′9″	19′3″	20′0″	23′6″
13-0	9-0	10-6	11-9	14-3	16-0	18-6	19-3	22-6
14-0		10-0	11-3	13-9	15-6	17-9	18-6	21-9
15-0		9-9	11-0	13-3	15-0	17-0	18-0	21-0
16-0		9-6	10-9	13-0	14-6	16-6	17-3	20-3
17-0		9-0	10-3	12-6	14-0	16-0	16-9	19-9
18-0			10-0	12-3	13-6	15-6	16-3	19-3
19-0			9-9	12-0	13-3	15-3	16-0	18-9
20-0			9-6	11-6	13-0	14-9	15-6	18-3
21-0			9-3	11-3	12-6	14-6	15-3	17-9

PIER BRACING

Supported Distance	Size, Wt. per Ft. and Details 1 Rod	STRUTS:—Sizes & Wts. per Ft. for Various Roadways				
		12′0″	14′0″	16′0″	18′0″	20′0″
25′	7/8″⌀ @ 2.6#/Ft. Details, 1-Rod 20#	2⊏ 4″×5¼ 17#/Ft.	2⊏ 4″×5¼ 17#/Ft.	2⊏ 4″×5¼ 17#/Ft.	2⊏ 4″×5¼ 17#/Ft.	2⊏ 4″×5¼ 17#/Ft.
50′	1⅛″⌀ @ 4.3#/Ft. Details, 1-Rod 30#	2⊏ 4″×5¼ 17#/Ft.	2⊏ 4″×5¼ 17#/Ft.	2⊏ 4″×5¼ 17#/Ft.	2⊏ 4″×5¼ 17#/Ft.	2⊏ 5″×6½ 19#/Ft.
75′	1⅜″⌀ @ 6.4#/Ft. Details, 1-Rod 45#	2⊏ 4″×5¼ 17#/Ft.	2⊏ 4″×5¼ 17#/Ft.	2⊏ 5″×6½ 19#/Ft.	2⊏ 5″×6½ 19#/Ft.	2⊏ 6″×8 22#/Ft.
100′	1⅝″⌀ @ 9.0#/Ft. Details, 1-Rod 65#	2⊏ 5″×6½ 19#/Ft.	2⊏ 5″×6½ 19#/Ft.	2⊏ 6″×8 22#/Ft.	2⊏ 6″×8 22#/Ft.	2⊏ 7″×9¾ 26#/Ft.
125′	1⅞″⌀ @ 12.0#/Ft. Details, 1-Rod 90#	2⊏ 6″×8 22#/Ft.	2⊏ 6″×8 22#/Ft.	2⊏ 6″×8 22#/Ft.	2⊏ 7″×9¾ 26#/Ft.	2⊏ 7″×9¾ 26#/Ft.

FIG. 16. STEEL TUBULAR PIERS FOR HIGHWAY BRIDGES, AMERICAN BRIDGE COMPANY.

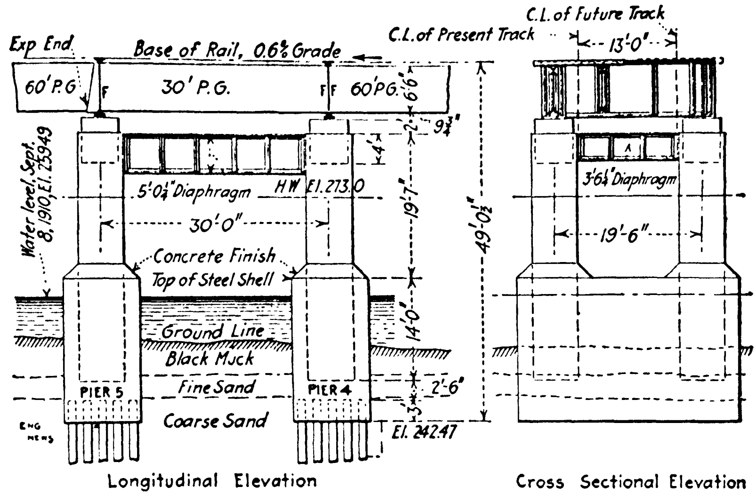


FIG. 17. STEEL TUBULAR PIERS, OXFORD MILL POND BRIDGE, CHICAGO & NORTHWESTERN RAILWAY.

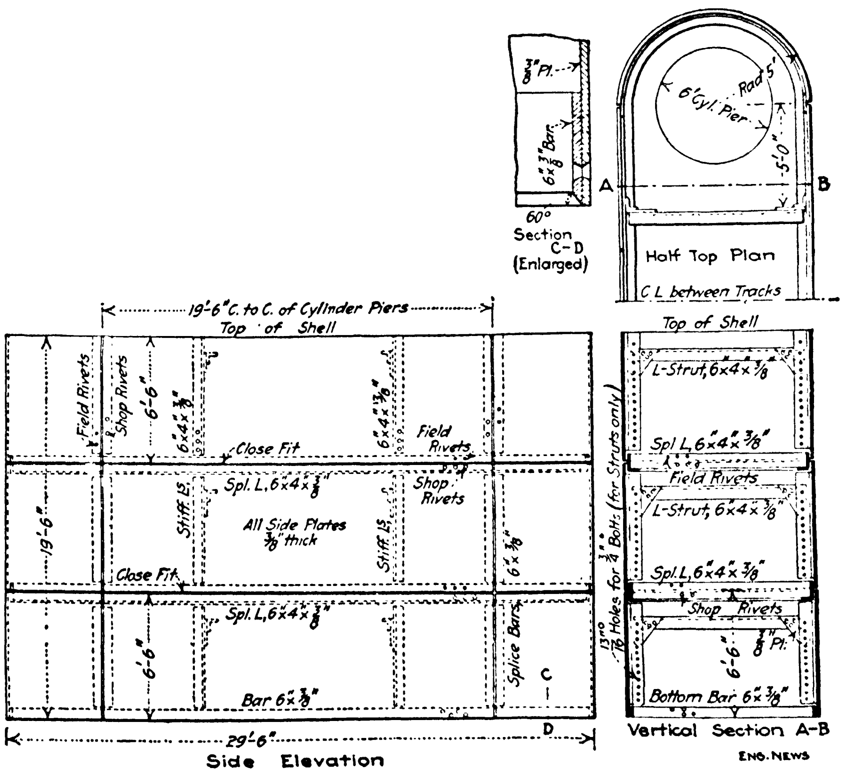


FIG. 18. STEEL SHELL FOR BASE OF CYLINDER PIERS OF THE OXFORD MILL POND BRIDGE, CHICAGO & NORTHWESTERN RAILWAY.

MASONRY AND CONCRETE DEFINITIONS AND SPECIFICATIONS.

Classification of Masonry.*

Kind.	Material.	Description.	Manner of Work.	Dressing. Joints or Beds.	Dressing. Face or Surface.
Bridge and Retaining Wall............	Stone.....	Dimension	Coursed	Smooth......	Smooth Rock-faced
		Ashlar....	Coursed Broken-coursed	Smooth Fine pointed Rough pointed	Smooth Rock-faced
		Rubble	Uncoursed	Rough pointed Scabbled	Rock-faced
	Concrete..	Reinforced Plain Rubble			
Arch...............	Stone.....	Ashlar	Coursed...	Smooth Fine pointed	Smooth Rock-faced
		Rubble	Uncoursed	Rough pointed Scabbled	Rock-faced
	Concrete..	Reinforced Plain			
	Brick	No. 1.....	English Bond Flemish Bond		
Culvert............	Stone.....	Rubble Dry	Uncoursed	Rough pointed Scabbled	Rock-faced
	Concrete..	Reinforced Plain Rubble			
Dry...............	Stone	Rubble	Uncoursed		

Definitions.*

Masonry, Bridge and Retaining Wall.—Masonry of stone or concrete, designed to carry the end of a bridge span or to retain the abutting earth, or both.

Masonry, Arch.—That portion of the masonry in the arch ring only, or between the intrados and the extrados.

Masonry, Culvert.—Flat-top masonry structure of stone or concrete, designed to sustain the fill above and to permit the free passage of water.

Masonry, Dry.—Masonry in which stones are built up without the use of mortar.

Concrete.

Concrete.—A compact mass of broken stone, gravel or other suitable material assembled together with cement mortar and allowed to harden.

Reinforced Concrete.—Concrete which has been reinforced by means of metal in some form, so as to develop the compressive strength of the concrete.

Rubble Concrete.—Concrete in which rubble stone are imbedded.

Brick.

Brick.—No. 1.—Hard burned brick, absorption not exceeding 2 per cent by weight.

Cement.

Cement.—A material of one of the three classes, Portland, Natural and Puzzolan, possessing the property of hardening into a solid mass when mixed with water.

* Adopted by Am. Ry. Eng. Assoc., Vol. 7, 1906, pp. 596–601, 619; Vol. 12, 1911.

Portland Cement.—This term shall be applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3 per cent has been made subsequent to calcination.

Natural Cement.—This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

Puzzolan Cement, as Made in North America.—An intimate mixture obtained by finely pulverizing together granulated basic blast furnace slag and slacked lime.

Courses and Bond.

Coursed.—Laid with continuous bed joints.

Broken Coursed.—Laid with parallel, but not continuous, bed joints.

Uncoursed.—Laid without regard to courses.

English Bond.—That disposition of bricks in a structure in which each course is composed entirely of headers or of stretchers.

Flemish Bond.—That disposition of bricks in a structure in which the headers and stretchers alternate in each course, the header being so placed that the outer end lies on the middle of a stretcher in the course below.

Dressing.

Dressing.—The finish given to the surface of stones or to concrete.

Smooth.—Having surface, the variations of which do not exceed one-sixteenth inch from the pitch line.

Fine Pointed.—Having irregular surface, the variations of which do not exceed one-quarter inch from the pitch line.

Rough Pointed.—Having irregular surface, the variations of which do not exceed one-half inch from the pitch line.

Scabbled.—Having irregular surface, the variations of which do not exceed three-quarters inch from the pitch line.

Rock-Faced.—Presenting irregular projecting face, without indications of tool mark.

Descriptive Words.

Abutment.—A supporting wall carrying the end of a bridge or span and sustaining the pressure of the abutting earth. The abutment of an arch is commonly called a bench wall.

Arris.—The external edge formed by two surfaces, whether plain or curved, meeting each other.

Ashlar.—A squared or cut block of stone with rectangular dimensions.

Backing.—That portion of a masonry wall or structure built in the rear of the face. It must be attached to the face and bonded with it. It is usually of a cheaper grade of work than the face.

Batter.—The slope or inclination of the face or back of a wall from a vertical line.

Bed.—The top and bottom of a stone. (See Course Bed; Natural Bed; Foundation Bed.)

Bed Joint.—A horizontal joint, or one perpendicular to the line of pressure.

Bench Wall.—The abutment from which an arch springs.

Bond.—The mechanical disposition of stone, brick or other building blocks by overlapping to break joints.

Build.—A vertical joint.

Centering.—A temporary support used in arch construction. (Also called centers.)

Clamp.—An instrument for lifting stone so designed that its grip on the surface of the stone is increased as the load is applied. That portion engaging the stone is of wood attached to a steel shoe, which in turn is hinged to the shank of the clamp in such a manner as to adjust itself to the surface of the body lifted.

Coping.—A top course of stone or concrete, generally slightly projecting, to shelter the masonry from the weather, or to distribute the pressure from exterior loading.

Course.—Each separate layer in stone, concrete or brick masonry.

Course Bed.—Stone, brick or other building material in position, upon which other material is to be laid.

Cramps.—Bars of iron having the ends turned at right angles to the body of the bar which enter holes in the upper side of adjacent stones.

Culvert.—A small covered passage for water under a roadway or embankment.

Dimension Stone.—(1) A block of stone cut to specified dimensions.

Dimension Stone.—(2) Large blocks of stone quarried to be cut to specified dimensions.

Dowels.—(a) Straight bars of iron which enter a hole in the upper side of one stone and also a hole in the lower side of the stone next above.

Dowel.—(b) A two-piece steel instrument used in lifting stone. The dowel engages the stone by means of two holes drilled into the stone at an angle of about 45 degrees pointing toward each other. The dowel is not keyed in place.

Draft.—A line on the surface of a stone cut to the breadth of the chisel.

Expansion Joint.—A vertical joint or space to allow for temperature changes.

Extrados.—The upper or convex surface of an arch.

Intrados.—The inner or narrow concave surface of an arch.

Face.—The exposed surface in elevation.

Facing.—In concrete: (1) A rich mortar placed on the exposed surfaces to make a smooth finish.

(2) Shovel facing by working the mortar of concrete to the face.

Final Set.—A stage of the process of setting marked by certain hardness. (See Cement Specifications.)

Flush.—(Adj.) Having the surface even or level with an adjacent surface.

Flush.—(Verb.) (1) To fill. (2) To bring to a level. (3) To force water to the surface of mortar or concrete by compacting or ramming.

Footing.—A projecting bottom course.

Form.—A temporary structure for giving concrete a desired shape.

Foundation.—(1) That portion of a structure usually below the surface of the ground, which distributes the pressure upon its support. (2) Also applied to the natural support itself; rock, clay, etc.

Foundation Bed.—The surface on which a structure rests.

Grout.—A mortar of liquid consistency which can easily be poured.

Header.—A stone which has its greatest length at right angles to the face of the wall, and which bonds the face stones to the backing.

Initial Set.—An early stage of the process of setting, marked by certain hardness. (See Cement Specifications.)

Joint.—The narrow space between adjacent stones, bricks or other building blocks, usually filled with mortar.

Lagging.—Strips used to carry and distribute the weight of an arch to the ribs or centering during its construction.

Lewis.—A four-piece steel instrument used in lifting stone. (The lewis engages the stone by means of a triangular-shaped hole into which it is keyed.)

Lock.—Any special device or method of construction used to secure a bond in the work.

Mortar.—A mixture of fine aggregate, cement or lime and water used to bind together the materials of concrete, stone or brick in masonry or to cover the surface of the same.

Natural Bed.—The surfaces of a stone parallel to its stratification.

Parapet.—A wall or barrier on the edge of an elevated structure for protection or ornament.

Paving.—Regularly placed stone or brick forming a floor.

Pier.—An intermediate support for arches or other spans.

Pitch.—(Verb.) To square a stone.

Pitched.—Having the arris clearly defined by a line beyond which the rock is cut away by the pitching chisel so as to make approximately true edges.

Pointing.—Filling joints or defects in the face of a masonry structure.

Retaining Wall.—A wall for sustaining the pressure of earth or filling deposited behind it.

Voussoirs.—The individual stones forming an arch. They are always of truncated wedge form.

Ring Stones.—The end voussoirs of an arch.

Riprap.—Rough stone of various sizes placed compactly or irregularly to prevent scour by water.

Rubble.—Field stone or rough stone as it comes from the quarry. When it is of a large or massive size it is termed block rubble.

Rubbed.—A fine finish made by rubbing with grit or sand stone.

Set.—(Noun) The change from a plastic to a solid or hard state.

Slope Wall.—A wall to protect the slope of an embankment or cut.

Soffit.—The under side of a projection.

Spall.—(Noun). A chip or small piece of stone broken from a large block.

Spandrel Wall.—The wall at the end of an arch above the springing line and extrados of the arch and below the coping or the string course.

Stretcher.—A stone which has its greatest length parallel to the face of the wall

Wing Wall.—An extension of an abutment wall to retain the adjacent earth.

SPECIFICATIONS FOR STONE MASONRY.*

General.

1. **Standard Specifications.**—The classification of masonry and the requirements for cement and concrete shall be those adopted by the American Railway Engineering Association.
2. **Engineer Defined.**—Where the term "Engineer" is used in these specifications, it refers to the engineer actually in charge of the work.

General Requirements.

3. **Stone.**—Stone shall be of the kinds designated and shall be hard and durable, of approved quality and shape, free from seams, or other imperfections. Unseasoned stone shall not be used where liable to injury by frost.
4. **Dressing.**—Dressing shall be the best of the kind specified.
5. Beds and joints or builds shall be square with each other, and dressed true and out of wind. Hollow beds shall not be permitted.
6. Stone shall be dressed for laying on the natural bed. In all cases the bed shall not be less than the rise.
7. Marginal drafts shall be neat and accurate.
8. Pitching shall be done to true lines and exact batter.
9. **Mortar.**—Mortar shall be mixed in a suitable box, or in a machine mixer, preferably of the batch type, and shall be kept free from foreign matter. The size of the batch and the proportions and the consistency shall be as directed by the engineer. When mixed by hand the sand and cement shall be mixed dry, the requisite amount of water then added and the mixing continued until the cement is uniformly distributed and the mass is uniform in color and homogeneous.
10. **Laying.**—The arrangement of courses and bond shall be as indicated on the drawings, or as directed by the engineer. Stone shall be laid to exact lines and levels, to give the required bond and thickness of mortar in beds and joints.
11. Stone shall be cleansed and dampened before laying.
12. Stone shall be well bonded, laid on its natural bed and solidly settled into place in a full bed of mortar.
13. Stone shall not be dropped or slid over the wall, but shall be placed without jarring stone already laid.
14. Heavy hammering shall not be allowed on the wall after a course is laid.
15. Stone becoming loose after the mortar is set shall be relaid with fresh mortar.
16. Stone shall not be laid in freezing weather, unless directed by the engineer. If laid, it shall be freed from ice, snow or frost by warming; the sand and water used in the mortar shall be heated.
17. With precaution, a brine may be substituted for the heating of the mortar. The brine shall consist of one pound of salt to eighteen gallons of water, when the temperature is 32 degrees Fahrenheit; for every degree of temperature below 32 degrees Fahrenheit, one ounce of salt shall be added.
18. **Pointing.**—Before the mortar has set in beds and joints, it shall be removed to a depth of not less than one (1) in. Pointing shall not be done until the wall is complete and mortar set; nor when frost is in the stone.
19 Mortar for pointing shall consist of equal parts of sand, sieved to meet the requirements, and Portland cement. In pointing, the joints shall be wet, and filled with mortar, pounded in with a "set-in" or calking tool and finished with a beading tool the width of a joint, used with a straight-edge.

Bridge and Retaining Wall Masonry—Ashlar Stone.

20. **Bridge and Retaining Wall Masonry. Ashlar Stone.**—The stone shall be large and well proportioned. Courses shall not be less than fourteen (14) in. or more than thirty (30) in. thick, thickness of courses to diminish regularly from bottom to top.
21. **Dressing.**—Beds and joints or builds of face stone shall be fine-pointed, so that the mortar layer should not be more than one-half (½) in. thick when the stone is laid.
22. Joints in face stone shall be full to the square for a depth equal to at least one-half the height of the course, but in no case less than twelve (12) in.

* Adopted by American Railway Engineering Association.

23. **Face or Surface.**—Exposed surfaces of the face stone shall be rock-faced, and edges pitched to the true lines and exact batter; the face shall not project more than three (3) in. beyond the pitch line.

24. Chisel drafts one and one-half (1½) in. wide shall be cut at exterior corners.

25. Holes for stone hooks shall not be permitted to show in exposed surfaces. Stone shall be handled with clamps, keys, lewis or dowels.

26. **Stretchers.**—Stretchers shall not be less than four (4) ft. long and have at least one and a quarter times as much bed as thickness of course.

27. **Headers.**—Headers shall not be less than four (4) ft. long, shall occupy one-fifth of face of wall, shall not be less than eighteen (18) in. wide in face, and, where the course is more than eighteen (18) in. high, width of face shall not be less than height of course.

28. Headers shall hold in heart of wall the same size shown in face, so arranged that a header in a superior course shall not be laid over a joint, and a joint shall not occur over a header; the same disposition shall occur in back of wall.

29. Headers in face and back of wall shall interlock when thickness of wall will admit.

30. Where the wall is three (3) ft. thick or less, the face stone shall pass entirely through. Backing shall not be permitted.

*31–a. **Backing.**—Backing shall be large, well-shaped stone, roughly bedded and jointed; bed joints shall not exceed one (1) in. At least one-half of the backing stone shall be of same size and character as the face stone and with parallel ends. The vertical joints in back of wall shall not exceed two (2) in. The interior vertical joints shall not exceed six (6) in. Voids shall be thoroughly filled with {*concrete.* / *spalls, fully bedded in cement mortar.*}

31–b. Backing shall be {*concrete.* / *headers and stretchers, as specified in paragraphs 26 and 27, and heart of wall filled with concrete.*}

32. Where the wall will not admit of such arrangement, stone not less than four (4) ft. long shall be placed transversely in heart of wall to bond the opposite sides.

33. Where stone is backed with two courses, neither course shall be less than eight (8) in. thick.

34. **Bond.**—Bond of stone in face, back and heart of wall shall not be less than twelve (12) in. Backing shall be laid to break joints with the face stone and with one another.

35. **Coping.**—Coping stone shall be full size throughout, of dimensions indicated on the drawings.

36. Beds, joints and top shall be fine-pointed.

37. Location of joints shall be determined by the position of the bed plates, and be indicated on the drawings.

38. **Locks.**—Where required, coping stone, stone in the wings of abutments, and stone on piers, shall be secured together with iron clamps or dowels, to the position indicated on the drawings.

BRIDGE AND RETAINING WALL MASONRY—RUBBLE STONE.

39. **Dressing.**—The stone shall be roughly squared, and laid in irregular courses. Beds shall be parallel, roughly dressed, and the stone laid horizontal to the wall. Face joints shall not be more than one (1) in. thick. Bottom stone shall be large, selected flat stone.

40. **Laying.**—The wall shall be compactly laid, having at least one-fifth the surface of back and face headers arranged to interlock, having all voids in the heart of the wall thoroughly filled with {*concrete.* / *suitable stones and spalls, fully bedded in cement mortar.*}

ARCH MASONRY—ASHLAR STONE.

41. **Arch Masonry, Ashlar Stone.**—Voussoirs shall be full size throughout and dressed true to templet, and shall have bond not less than thickness of stone.

42. **Dressing.**—Joints of voussoirs and intrados shall be fine-pointed. Mortar joints shall not exceed three-eighths (⅜) in.

43. **Face or Surface.**—Exposed surface of the ring stone shall be {*smooth.* / *rock faced, with a marginal draft.*}

44. Number of courses and depth of voussoirs shall be indicated on the drawings.

45. Voussoirs shall be placed in the order indicated on the drawings.

* Paragraphs 31–a and 31–b are so arranged that either may be eliminated according to requirements. Optional clauses printed in italics.

46. **Backing.**—Backing shall consist of { *concrete.* / *large stone, shaped to fit the arch bonded to the spandrel and laid in full bed of mortar.* }

47. Where waterproofing is required, a thin coat of mortar or grout shall be applied evenly for a finishing coat, upon which shall be placed a covering of approved waterproofing material.

48. Centers shall not be struck until directed by the engineer.

49. **Bench Walls, Piers, Spandrels, etc.**—Bench walls, piers, spandrels, parapets, wing walls and copings shall be built under the specifications for Bridge and Retaining Wall Masonry, Ashlar Stone.

Arch Masonry—Rubble Stone.

50. **Arch Masonry, Rubble Stone.**—Voussoirs shall be full size throughout, and shall have bond not less than thickness of voussoirs.

51. **Dressing.**—Beds shall be roughly dressed to bring them to radial planes.

52. Mortar joints shall not exceed one (1) in.

53. **Face or Surface.**—Exposed surfaces of the ring stone shall be rock-faced, and edges pitched to true lines.

54. Voussoirs shall be placed in the order indicated on the drawings.

55. **Backing.**—Backing shall consist of { *concrete.* / *large stone, shaped to fit the arch, bonded to the spandrel, and laid in full bed of mortar.* }

56. Where waterproofing is required, a thin coat of mortar or grout shall be applied evenly for a finishing coat, upon which shall be placed a covering of approved waterproofing material.

57. Centers shall not be struck until directed by the engineer.

58. **Bench Walls, Piers, Spandrels, etc.**—Bench walls, piers, spandrels, parapets, wing walls and copings shall be built under the specifications for Bridge and Retaining Wall Masonry, Rubble Stone.

Culvert Masonry.

59. **Culvert Masonry.**—Culvert Masonry shall be laid in cement mortar. Character of stone and quality of work shall be the same as specified for Bridge and Retaining Wall Masonry, Rubble Stone.

60. **Side Walls.**—One-half the top stone of the side walls shall extend entirely across the wall.

61. **Cover Stones.**—Covering stone shall be sound and strong, at least twelve (12) in. thick, or as indicated on the drawings. They shall be roughly dressed to make close joints with each other, and lap their entire width at least twelve (12) in. over the side walls. They shall be doubled under high embankments, as indicated on the drawings.

62. **End Walls, Coping.**—End walls shall be covered with suitable coping, as indicated on the drawings.

Dry Masonry.

63. **Dry Masonry.**—Dry Masonry shall include dry retaining walls and slope walls.

64. **Retaining Walls.**—Retaining Walls and Dry Masonry shall include all walls in which rubble stone laid without mortar is used for retaining embankments or for similar purposes.

65. **Dressing.**—Flat stone at least twice as wide as thick shall be used. Beds and joints shall be roughly dressed square to each other and to face of stone.

66. Joints shall not exceed three-quarters (¾) in.

67. **Disposition of Stone.**—Stone of different sizes shall be evenly distributed over entire face of wall, generally keeping the larger stone in lower part of wall.

68 The work shall be well bonded and present a reasonably true and smooth surface, free from holes or projections.

69. **Slope Walls.**—Slope Walls shall be built of such thickness and slope as directed by the engineer. Stone shall not be used in this construction which does not reach entirely through the wall. Stone shall be placed at right angles to the slopes. The wall shall be built simultaneously with the embankment which it is to protect.

SPECIFICATIONS FOR CONCRETE, PLAIN AND REINFORCED.

American Railway Engineering Association.

Adopted 1920.

I. Material.

1. **Cement.**—The cement shall meet the requirements of the American Railway Engineering Association's "Specifications for Portland Cement." It shall be stored in a weather-tight structure with the floor raised not less than one foot from the ground in such a manner as to permit easy access for proper inspection and identification of each shipment. Cement that has hardened or partially set shall not be used.

2. **Fine Aggregate.**—(a) The fine aggregate shall consist of sand, crushed stone or gravel screenings, graded from fine to coarse, and passing when dry, a screen having holes one-quarter (¼) inch in diameter. Not more than twenty-five (25) per cent. by weight shall pass a No. 50 sieve, and not more than six (6) per cent. a No. 100 sieve when screened dry, nor more than ten (10) per cent. dry weight shall pass a No. 100 sieve when washed on the sieve with a stream of water. It shall be clean and free from soft particles, mica, lumps of clay, loam or organic matter.

(b) The fine aggregate shall be of such quality that mortar briquettes made of one (1) part of Portland cement and three (3) parts of the fine aggregate by weight shall show a tensile strength, after an age of seven (7) days, not less than the strength of briquettes of the same age, made of mortar of the same consistency in the proportion of one (1) part of the same cement to three (3) parts of standard Ottawa sand.

3. **Coarse Aggregate.**—The coarse aggregate shall consist of gravel or crushed stone, which, unless otherwise specified or called for on the plans, shall, for plain mass concrete, pass a screen having holes two and one-quarter (2¼) inches in diameter, and for reinforced concrete a screen having holes one and one-quarter (1¼) inches in diameter; and be retained on a screen having holes one-fourth (¼) inch in diameter, and shall be graded in size from the smallest to the largest particles. It shall be clean, hard, durable and free from all deleterious matter; coarse aggregate containing dust, soft or elongated particles shall not be used.

4. **Stone for Rubble or Cyclopean Concrete.**—These stones shall be of good quality, clean, dense and hard, without seams and having sharp edges. They shall not be smaller than of a size known as "one-man stone."

5. **Slag.**—Provided the contract specifically permits the use of crushed slag as a coarse aggregate, it shall be air cooled, blast furnace slag, conforming to all the requirements for coarse aggregate specified in Paragraph 3. The crushed slag shall weigh not less than seventy (70) lb. per cubic foot, and shall be obtained only from such banks as have the approval of the Engineer. All slag used shall have seasoned in the bank for a period not less than one (1) year, unless in the opinion of the Engineer a shorter period is sufficient.

6. **Water.**—The water shall be free from oil, acid and injurious amounts of vegetable matter, alkalies or other salts.

7. **Steel Reinforcement.**—(a) All structural steel shapes used for reinforcing shall conform to the requirements of the American Railway Engineering Association's "Specifications for Steel Railway Bridges."

(b) All steel rods or bars used for reinforcing shall conform to the requirements of the American Railway Engineering Association's "Specifications for Billet-Steel Concrete Reinforcement Bars."

II. Proportioning.

8. **Unit of Measure.**—The unit of measure shall be the cubic foot. Ninety-four (94) lb. (one (1) sack or one-fourth (¼) barrel) of cement shall be assumed as one (1) cubic foot.

9. **Proportions.**—(a) The proportions of the materials shall be in accordance with the plans, or detailed specifications, or schedule governing the work. When not otherwise specified, the proportions by volume shall be as follows: (See 8, 10.)

Class.	*Use.*	*Cement.*	*Fine Aggregate.*	*Coarse Aggregate.*
A	Reinforced concrete—Concrete deposited under water..	1	2	4
B	Mass concrete in forms..........................	1	2½	5
C	Foundation......................................	1	3	6

(b) Rubble or cyclopean concrete, when permitted by the contract, shall be either Class " B " or Class " C " concrete, having embedded in it large stones.

(c) For any given class of concrete, the relative proportion of cement to fine aggregate shall not be modified. The relative proportion of fine to coarse aggregate shall be modified, if necessary, during the progress of the work, so as to obtain the maximum density. (See §9a.)

10. **Measuring Proportions.**—The various ingredients, including the water, shall be measured separately, and the methods of measurement shall be such as to invariably secure the proper proportions. The fine and coarse aggregate shall be measured loosely as thrown into the measuring receptacle. (See 8, 9a.)

11. **Consistency.**—The quantity of water used in mixing shall be the least amount that will produce a plastic or workable mixture which can be worked into the forms and around the reinforcement. Under no circumstances shall the consistency of the concrete be such as to permit a separation of the coarse aggregate from the mortar in handling. An excess of water will not be permitted, as it seriously affects the strength of the concrete and any batch containing such an excess will be rejected.

12. **Premixed Aggregate.**—(a) Provided the contract specifically permits, premixed aggregate may be used instead of separate fine and coarse aggregates. Frequent tests shall be made to determine the relative proportions of fine and coarse aggregates, and if these proportions are unsatisfactory to the Engineer, or so irregular as to make it impracticable to secure a properly proportioned concrete, he may reject the material, or require that it be screened and used as separate fine and coarse aggregates.

(b) The proportion of the cement to the fine aggregate shall at no time be less than that specified for the classes of concrete where separate aggregates are used. (See §9a.)

III. Forms.

13. **Materials.**—(a) The forms shall be of wood or metal, and shall conform to the shape, lines and dimensions of the concrete as called for on the plans. Form lumber used against the concrete shall be dressed on one side and both edges, to a uniform thickness and width, and shall be sound and free of loose knots.

(b) For all exposed edges, corners or other projections of the concrete, suitable moldings or bevels shall be placed in the angles of the forms to round or bevel the edges of the concrete.

14. **Workmanship.**—(a) The forms shall be well built, substantial and unyielding, and made sufficiently tight to prevent leakage of mortar and voids in the concrete. They shall be properly braced or tied together by rods, bolts or wires. Metal braces or ties shall be so arranged that when the forms are removed, no metal shall be within one (1) inch of the face of the finished work.

(b) The face forms shall be securely fastened to the studding or uprights in horizontal lines.

(c) Any irregularities in the forms which may mar the exposed surface of the concrete shall be removed or filled.

15. **Inspection.**—Where necessary, temporary openings shall be provided at the base of the forms to facilitate cleaning and inspection directly before placing concrete. (See §23b.)

16. **Oiling.**—The inside of the forms shall generally be coated with raw paraffin or other non-staining mineral oil; or thoroughly wet with water, except in freezing weather. (See §23b).

17. **Removal of Forms.**—The forms shall not be removed until authorized by the Engineer.

IV. Reinforcement.

18. **Placing Reinforcement.**—Reinforcing steel shall be cleaned of all mill and rust scales before being placed in the forms. All reinforcement shall be placed in its proper position as required by the plans and securely wired or fastened in place, well in advance of the concreting, and shall be inspected and approved by the Engineer before any concrete is deposited. (See 23b.)

19. **Splicing Reinforcement.**—Wherever it is necessary to splice the reinforcement otherwise than as shown on the plans, the character of the splice shall be decided by the Engineer on the basis of safe bond stress and the stress in the reinforcement at the point of splice. Splices shall not be made at points of maximum stress.

V. Mixing.

20. **Machine Mixing.**—(a) All concrete shall be mixed by machine (except when under special conditions the Engineer permits otherwise), in a batch mixer of an approved type, equipped with suitable charging hopper, water storage and a water measuring device which can be locked.

(b) The ingredients of the concrete shall be mixed to the required consistency and the mixing continued not less than one and one-half ($1\frac{1}{2}$) minutes after all the materials are in the mixer, and before any part of the batch is discharged. The mixer shall be completely emptied before receiving materials for the succeeding batch. The volume of the mixed material used per batch shall not exceed the manufacturers' rated capacity of the drum. (See § 11.)

21. **Hand Mixing.**—When it is permitted to mix by hand, the mixing shall be done on a watertight platform of sufficient size to accommodate men and materials for the progressive and rapid mixing of at least two batches of concrete at the same time. The batches shall not exceed one-half (½) cubic yard each. The materials shall be mixed dry until the mixture is of a uniform color, the required amount of water added, and the mixing continued until the batch is of a uniform consistency and character throughout. Hand mixing will not be permitted for concrete deposited under water. (See § 11.)

22. **Retempering.**—The retempering of mortar or concrete which has partially hardened; that is, remixing with or without additional materials or water, will not be permitted.

VI. Depositing.

23. **General.**—(a) Before beginning a run of concrete, all hardened concrete or foreign materials shall be completely removed from the inner surfaces of all conveying equipment.

(b) Before depositing any concrete, all debris shall be removed from the space to be occupied by the concrete, all steel reinforcing shall be secured in its proper location, all forms shall be thoroughly wetted except in freezing weather unless they have been previously oiled, and all form work and steel reinforcing shall be inspected and approved by the Engineer. (See § 15, 16 and 18.)

24. **Handling.**—Concrete shall be handled from the mixer to the place of final deposit as rapidly as possible, and by methods of transporting which shall prevent the separation of the ingredients. The concrete shall be deposited directly into the forms as nearly as possible in its final position so as to avoid rehandling. The piling up of concrete material in the forms in such manner as to permit the escape of mortar from the coarse aggregate will not be permitted. Under no circumstances shall concrete that has partially set be deposited in the work. (See § 22.)

25. **Compacting.**—During and after depositing, the concrete shall be compacted by means of a shovel or other suitable tool moved up and down continuously in the concrete until it has all settled into place and water has flushed to the surface. The concrete shall be thoroughly worked around all reinforcing material so as to completely surround and embed the same.

26. **Cold Weather.**—During cold weather, the concrete at the time it is mixed and deposited in the work shall have a temperature not lower than fifty (50) degrees Fahrenheit, and suitable means shall be provided to maintain this temperature for at least seventy-two (72) hours thereafter, and until the concrete has thoroughly set. The methods of heating materials and protecting the concrete shall be approved by the Engineer. The use of any salt or chemical to prevent freezing will not be permitted.

27. **Depositing on or Against Set Concrete.**—Before depositing new concrete on or against concrete which has set, the forms shall be retightened against the face of the latter, the surface of the set concrete shall be roughened and thoroughly cleaned of foreign matter and laitance, and saturated with water. The new concrete placed in contact with set or partially set concrete shall contain an excess of mortar to insure bond. To insure this excess of mortar at the juncture of the set and newly deposited concrete on vertical or inclined surfaces, the cleaned and drenched surface of the set concrete shall first be slushed with a coating of mortar, not less than one inch thick, composed of one (1) part cement to two (2) parts fine aggregate, against which the new concrete shall be deposited before this mortar has had time to attain its initial set.

28. **Rubble or Cyclopean Concrete.**—After each layer of concrete is placed, and before it has taken its initial set, the stones are to be thoroughly bedded in the soft concrete. No stone shall be placed nearer than one (1) foot to any finished surface; nor nearer than six (6) inches to any adjacent stone. After the stones are in place another layer of concrete shall be placed sufficient to cover the stones to a depth of at least six (6) inches.

When stratified stones are used, they shall be laid upon their natural bed. (See § 4, 9b.)

VII. Depositing Concrete Under Water.

29. **General.**—Concrete shall not be deposited in water without the written consent of the Engineer. A written statement of the methods and plans of equipment to be used shall be submitted to and approved by the Engineer before the work is started. (See §9a, 11, 21.)

30. **Cofferdams.**—Cofferdams shall be sufficiently tight to prevent any current through the space in which the concrete is to be deposited. Pumping will not be permitted while the concrete is being deposited, nor until it has fully set.

31. **Method.**—The concrete shall be deposited by such method as will prevent the washing of the cement from the mixture. In no case shall the concrete be allowed to fall through the water.

32. **Tremie.**—The tremie, where used, shall be about fourteen (14) or sixteen (16) inches in diameter, and made flanged and put together with gaskets. The initial filling of the tremie shall be done in such manner as not to permit the concrete to drop through the water. It shall be kept filled at all times, and the discharge end raised a few inches at a time as the filling progresses.

The greatest care shall be used to prevent the charge being lost in moving the tremie about on the surface of the deposited concrete. In case the charge is lost, the tremie must be withdrawn and refilled.

33. **Drop Bottom Bucket.**—(a) The bucket, where used, shall be of such a type that it cannot be dumped until it rests on the surface upon which the concrete is to be deposited. The frame shall extend below the closed bottom doors so they may open freely downward and outward when tripped. The ends of the bucket shall extend without openings to the bottom of the frame. The top of the bucket shall be open.

(b) The bucket shall be completely filled, and slowly lowered to avoid unnecessary back wash. When discharged the bucket shall be withdrawn slowly until clear of the concrete.

34. **Bagging.**—The bags, when used, shall be of jute or other coarse cloth. They shall be about two-thirds filled with concrete, and shall be carefully placed by hand in a header and stretcher system so the whole mass is interlocked

35. **Continuous Operation.**—Where possible, the concrete shall be deposited continuously from the time the work is started until it is brought above water level or to the finished surface. The work shall be carried on with sufficient rapidity to insure bonding of the successive layers. The surface of the deposited concrete shall be kept as nearly level as possible.

36. **Laitance.**—Great care shall be exercised to disturb the concrete as little as possible while it is being deposited, to avoid the formation of laitance. On completing a section of concrete, the laitance shall be entirely removed after the concrete has thoroughly set and before the work is resumed.

VIII. Joints.

37. **General.**—(a) Instructions given on the plans, in the detailed specifications or schedule governing the work as to location and construction of joints, shall be strictly followed.

(b) When the structures or portions of the structures are designed to be monolithic, they shall be cast integrally, except as hereinafter modified. (See § 38a, b, c, d.)

38. **Construction Joints.**—(a) When necessary to provide construction joints not indicated, or specified, such joints shall be located and formed so as to least impair the strength and appearance of the structure. Where conditions require, the joints shall be reinforced as directed by the Engineer, in order to secure the necessary bond strength.

(b) Horizontal construction joints shall be prepared at the time the work is interrupted by thoroughly roughening the surface and providing keys by embedding stones which project above the surface, or mortises by embedding timbers which shall be removed before the work of placing concrete is resumed.

(c) At all horizontal or vertical construction joints, the surface of the previously deposited concrete shall always be roughened and cleaned of all laitance and foreign material before depositing new concrete. (See § 27.)

(d) Where girders, beams and slabs are designed to be monolithic with walls and columns, they shall not be cast until four (4) hours after the completion of the walls or columns in order to permit of shrinkage or settlement. In case the columns are structural steel, encased in concrete or concrete columns having flaring heads, the lapse of time to allow for shrinkage or settlement need not be observed. (See §37b.)

39. **Watertight Joints.**—When it is not possible to finish a complete section in one continuous operation, and a watertight joint is required, sheet lead or other metal, not less than six inches wide, and extending the full length of the joint, shall be embedded equally in the two deposits of concrete.

40. **Sliding Joints.**—Where sliding joints are to be provided, the seat shall be finished with a smooth trowel surface and shall not have the superimposed concrete placed upon it until the previously deposited concrete has thoroughly set. Unless otherwise indicated on the plans, or specified, two thicknesses of building paper shall be placed over the bearing before the superimposed concrete is deposited, in order to make a defined sliding joint.

41. **Expansion Joints.**—(a) At all expansion joints, the break in the bond between the two sections shall be complete, and shall be insured by the application of petroleum oil, hot coal tar pitch, tarred felt or similar material over the entire joint surface of the first deposited concrete.

(b) No reinforcement shall extend across an expansion joint.

(c) Triangular shaped grooves shall be formed in the exposed surface of the concrete at all expansion joints in walls or abutments.

(d) Where expansion joints are formed between two distinct concrete members, and said joint is exposed, it shall be filled with an elastic joint filler of approved quality.

IX. Surfacing and Finishing.

42. **General.**—Except where a special surface or finish is required, the surfacing and finishing shall be done in accordance with the requirements specified for a "Spaded Surface." (See §43a, b, c.)

43. **Spaded Surface.**—(a) The coarse aggregate shall be carefully worked back from the forms into the mass of the concrete with spades, fine stone forks, bars or other suitable tools, so as to bring a surface of mortar against the form. Care shall be taken to remove all air pockets and to prevent voids in the surface.

(b) Except where otherwise directed by the Engineer, face forms shall be removed as soon as the setting of the concrete will permit. (See §17.)

(c) After the removal of the forms, any holes or voids in the surface of the concrete shall be filled with a mortar made of the same proportions of sand and cement as those of the concrete and rubbed smooth and even with the surface with a wooden float. A trowel shall not be used for this purpose. (See § 42.)

44. **Top Surfaces.**—(a) Top surfaces shall generally be "struck" with a straight edge or "floated" after the coarse aggregates have been forced below the surface.

(b) Where "sidewalk finish" is called for on the plans, it shall be made by the spreading of a 1 : 2 mortar at least three-quarters (¾) inch thick, and floating this to a smooth surface. This finishing coat shall be put on before the concrete has taken its initial set. For a walk, the surface shall be slightly roughened with a special tool or by sweeping with a coarse broom.

45. **Wetting Surfaces.**—The surfaces of concrete exposed to premature drying shall be kept thoroughly and constantly wetted for a period of at least three (3) days. For wearing surfaces, this period shall be at least ten (10) days.

SPECIFICATIONS FOR BILLET-STEEL CONCRETE REINFORCEMENT BARS.

Adopted 1920.

1. **Material Covered.**—(a) These specifications cover two classes of billet-steel concrete reinforcement bars, namely: plain and deformed.

(b) Plain and deformed bars are of three grades, namely: structural-steel, intermediate and hard.

(c) Twisted bars will not be accepted under these specifications.

2. **Basis of Purchase.**—The structural-steel grade shall be used unless otherwise specified.

(Specifications for manufacture and tests comply with the A. S. T. M. Specifications for Billet-Steel Reinforcement Bars, Chapter XV.)

REFERENCES.—Plain masonry and concrete abutments and piers, only, have been considered in this chapter. The following books may be consulted for additional information.

Baker's "Masonry Construction," John Wiley & Sons, gives a full discussion of the design of masonry, plain and reinforced concrete abutments and piers, and the different methods of constructing abutments and piers.

Fowler's "Ordinary Foundations," John Wiley & Sons, gives a full discussion of the design and construction of abutments and pi rs, with special attention given to the coffer dam process.

Jacoby and Davis' "Foundations of Bridges and Buildings," McGraw-Hill Book Co., gives a full discussion of the design and construction of abutments and piers.

Bulletin 140 of the Am. Ry. Eng. Assoc. has an article on the Design of Railway Bridge Abutments by Mr. J. H. Prior, Asst. Engineer, C. M. & St. P. Ry. This article describes in detail the standard plain and reinforced concrete abutments used by the C. M. & St. P. Ry.

Williams' "Design of Masonry Structures and Foundations," McGraw-Hill Book Co., gives a full discussion of the design and construction of masonry structures and foundations.

CHAPTER VII.

Timber Bridges and Trestles.

Definitions.—The following definitions have been adopted by the American Railway Engineering Association.

Wooden Trestle.—A wooden structure composed of upright members supporting simple horizontal members or beams, the whole forming a support for loads applied to the horizontal members.

Frame Trestle.—A structure in which the upright members or supports are framed timbers.

Pile Trestle.—A structure in which the upright members or supports are piles.

Bent.—The group of members forming a single vertical support of a trestle, designated as pile bent where the principal members are piles, and as framed bent where of framed timbers.

Post.—One of the vertical or battered members of the bent of a framed trestle.

Pile.—(See definition under subject of Piles and Pile Driving.)

Batter.—A deviation from the vertical in upright members of a bent.

Cap.—A horizontal member upon the top of piles or posts, connecting them in the form of a bent.

Sill.—A lower horizontal member of a framed bent.

Sub-Sill.—A timber bedded in the ground to support a framed bent.

Intermediate Sill.—A horizontal member in the plane of the bent between the cap and sill to which the posts are framed.

Sway Brace.—A member bolted or spiked to the bent and extending diagonally across its face.

Longitudinal Strut or Girt.—A stiff member running horizontally, or nearly so, from bent to bent.

Longitudinal X-Brace.—A member extending diagonally from bent to bent in a vertical or battered plane.

Sash Brace.—A horizontal member secured to the posts or piles of a bent.

Stringer.—A longitudinal member extending from bent to bent and supporting the ties.

Jack Stringer.—A stringer placed outside of the line of main stringers.

Tie.—A transverse timber resting on the stringers and supporting the rails.

Guard Rail.—A longitudinal member, usually a metal rail, secured on top of the ties inside of the track rail, to guide derailed car wheels.

Guard Timber.—A longitudinal timber framed over the ties outside of the track rail, to maintain the spacing of the ties.

Packing Block.—A small member, usually wood, used to secure the parts of a composite member in their proper relative positions.

Packing Spool or Separator.—A small casting used in connection with packing bolts to secure the several parts of a composite member in their proper relative positions.

Drift Bolt.—A piece of round or square iron of specified length, with or without head or point, driven as a spike.

Dowel.—An iron or wooden pin, extending into, but not through, two members of the structure to connect them.

Shim.—A small piece of wood or metal placed between two members of a structure to bring them to a desired relative position.

Fish-Plate.—A short piece lapping a joint, secured to the side of two members, to connect them end to end.

Bulkhead.—A wall of timber placed against the side of an end bent to retain the embankment.

Structural Timber.

Definitions.—The following definitions have been adopted by the American Railway Engineering Association.

Timber.—A single stick of wood of regular cross-section.

Cross-Section.—A section of a stick at right angles to the axis.

True.—Of uniform cross-section. Defects are caused by wavy or jagged sawing or consist of trapezoidal instead of rectangular cross-sections.

Axis.—The line connecting the centers of successive cross-sections of a stick.

Straight.—Having a straight line for an axis.

Out of Wind.—Having the longitudinal surfaces plane.

Full Length.—Long enough to "square" up to the length specified in the order.

Corner.—The line of intersection of the planes of two adjacent longitudinal surfaces.

Girth.—The perimeter of a cross-section.

Side.—Either of the two wider longitudinal surfaces of a stick.

Edge.—Either of the two narrower longitudinal surfaces of a stick.

Face.—The surface of a stick which is exposed to view in the finished structure.

Sapwood.—A cylinder of wood next to the bark and of lighter color than the wood within. It may be of uneven thickness.

Heartwood.—The older and central part of a log, usually darker in color than sapwood. It appears in strong contrast to the sapwood in some species, while in others it is but slightly different in color.

Springwood.—The inner part of the annual ring formed in the earlier part of the season, not necessarily in the spring, and often containing vessels or pores.

Summerwood.—The outer part of the annual ring formed later in the season, not necessarily in the summer, being usually dense in structure and without conspicuous pores.

Decay.—Complete or partial disintegration of the cell walls, due to the growth of fungi.

Sound.—Free from decay.

Solid.—Without cavities; free from loose heart, wind shakes, bad checks, splits or breaks, loose slivers, and worm or insect holes.

Wane.—A deficient corner due to curvature or to taper of the log.

Square Cornered.—Free from wane.

Knot.—The hard mass of wood formed in a trunk at a branch, with the grain distinct and separate from the grain of the trunk.

Cross-Grain.—The gnarly mass of wood surrounding a knot, or grain injuriously out of parallel with the axis.

Wind Shake.—A crack or fissure, or a series of them, caused during growth.

STANDARD DEFECTS OF STRUCTURAL TIMBER.*

The standard defects included in the following list are mostly such as may be termed natural defects, as distinguished from defects of manufacture. The latter have usually been omitted, because the defects of manufacture are of minor significance in the grading of structural timber:

Sound Knot.—A sound knot is one which is solid across its face and is as hard as the wood surrounding it. It may be either red or black, and is so fixed by growth or position that it will retain its place in the piece.

Loose Knot.—A loose knot is one not firmly held in place by growth or position.

Pith Knot.—A pith knot is a sound knot with a pith hole not more than ¼ in. in diameter† in the center.

Encased Knot.—An encased knot is one which is surrounded wholly or in part by bark or pitch. Where the encasement is less than ⅛ in. in width on each side, nor exceeding one-half the circumference of the knot, it shall be considered a sound knot.

Rotten Knot.—A rotten knot is one not as hard as the wood surrounding it.

Pin Knot.—A pin knot is a sound knot not over ½ in. in diameter.

Standard Knot.—A standard knot is a sound knot not over 1½ in. in diameter.

Large Knot.—A large knot is a sound knot, more than 1½ in. in diameter.

Round Knot.—A round knot is one which is oval or circular in form.

Spike Knot.—A spike knot is one sawn in a lengthwise direction. The mean or average diameter shall be taken as the size of these knots.

Pitch Pockets.—Pitch pockets are openings between the grain of the wood, containing more or less pitch or bark. These shall be classified as small, standard and large pitch pockets.

Small Pitch Pocket.—(a).—A small pitch pocket is one not over ⅛ in. wide.

Standard Pitch Pocket.—(b).—A standard pitch pocket is one not over ⅜ in. wide nor over 3 in. in length.

Large Pitch Pocket.—(c).—A large pitch pocket is one over ⅜ in. wide, or over 3 in. in length.

Pitch Streak.—A pitch streak is a well-defined accumulation of pitch at one point in the piece. When not sufficient to develop a well-defined streak, or where the fiber between grains, that is, the coarse grained fiber, usually termed "spring wood," is not saturated with pitch, it shall not be considered a defect.

* Adopted by Am. Ry. Eng. Assoc., Vol. 8, 1907.

† Measurements which refer to the diameter of knots or holes shall be considered as the mean or average diameter in all cases.

Shakes.—Shakes are splits or checks in timber which usually cause a separation of the wood between annual rings.

Ring Shake.—An opening between annual rings.

Through Shakes.—A shake which extends between two faces of a timber.

Rot, Dote and Red Heart.—Any form of decay which may be evident either as a dark red discoloration not found in the sound wood, or by the presence of white or red rotten spots, shall be considered as a defect.

Wane.—(See definition under the subject of Structural Timber.)

Note.—See additional definitions of defects under Structural Timber.

Piles and Pile Driving.*

The following definitions and the principles of Pile Driving have been adopted by the American Railway Engineering Association.

Pile.—A member usually driven or jetted into the ground and deriving its support from the underlying strata, and by the friction of the ground on its surface. The usual functions of a pile are: (a) to carry a superimposed load; (b) to compact the surrounding ground; (c) to form a wall to exclude water and soft material, or to resist the lateral pressure of adjacent ground.

Head of Pile.—The upper end of a pile.

Foot of Pile.—The lower end of a pile.

Butt of Pile.—The larger end of a pile.

Tip of Pile.—The smaller end of a pile.

Bearing Pile.—One used to carry a superimposed load.

Screw Pile.—One having a broad-bladed screw attached to its foot to provide a larger bearing area.

Disc Pile.—One having a disc attached to its foot to provide a larger bearing area.

Batter Pile.—One driven at an inclination to resist forces which are not vertical.

Sheet Pile.—Piles driven in close contact in order to provide a tight wall, to prevent leakage of water and soft materials, or driven to resist the lateral pressure of adjacent ground.

Pile Driver.—A machine for driving piles.

Hammer.—A weight used to deliver blows to a pile to secure its penetration.

Drop Hammer.—One which is raised by means of a rope and then allowed to drop.

Steam Hammer.—One which is automatically raised and dropped a comparatively short distance by the action of a steam cylinder and piston supported in a frame which follows the pile.

Leads.—The upright parallel members of a pile driver which support the sheaves used to hoist the hammer and piles, and which guide the hammer in its movement.

Cap.—A block used to protect the head of a pile and to hold it in the leads during driving.

Ring.—A metal hoop used to bind the head of a pile during driving.

Shoe.—A metal protection for the point or foot of a pile.

Follower.—A member interposed between the hammer and a pile to transmit blows to the latter when below the foot of the leads.

PILE-DRIVING—Principles of Practice.—(1) A thorough exploration of the soil by borings, or preliminary test piles, is the most important prerequisite to the design and construction of pile foundations.

(2) The cost of exploration is frequently less than that otherwise required merely to revise the plans of the structure involved, without considering the unnecessary cost of the structures due to lack of information.

(3) Where adequate exploration is omitted, it may result in the entire loss of the structure, or in greatly increased cost.

(4) The proper diameter and length of pile, and the method of driving depend upon the result of the previous exploration and the purpose for which they are intended.

(5) Where the soil consists wholly or chiefly of sand, the conditions are most favorable to the use of the water jet.

(6) In harder soils containing gravel the use of the jet may be advantageous, provided sufficient volume and pressure be provided.

(7) In clay it may be economical to bore several holes in the soil with the aid of the jet before driving the pile, thus securing the accurate location of the pile, and its lubrication while being driven.

(8) In general, the water jet should not be attached to the pile, but handled separately.

(9) Two jets will often succeed where one fails; in special cases a third jet extending a part of the depth aids materially in keeping loose the material around the pile.

(10) Where the material is of such a porous character that the water from the jets may be

* For an elaborate bibliography on "Piles and Pile Driving" see Am. Ry. Eng. Assoc., Vol. 10.

dissipated and fail to come up in the immediate vicinity of the pile, the utility of the jet is uncertain, except for a part of the penetration.

(11) A steam or drop hammer should be used in connection with the water jet, and used to test the final rate of penetration.

(12) The use of the water jet is one of the most effective means of avoiding injury to piles by overdriving.

(13) There is danger from overdriving when the hammer begins to bounce. Overdriving is also indicated by the bending, kicking or staggering of the pile.

(14) The brooming of the head of a pile dissipates a part, and in some cases all, of the energy due to the fall of the hammer.

(15) The weight or the drop of the hammer should be proportioned to the weight of the pile, as well as to the character of the soil to be penetrated.

(16) The steam hammer is more effective than the drop hammer in securing the penetration of a pile without injury, because of the shorter interval between blows.

(17) Where shock to surrounding material is apt to prove detrimental to the structure, the steam hammer should always be used instead of the drop hammer. This is especially true in the case of sheet piling which is intended to prevent the passage of water. In some cases also the jet should not be used.

(18) In general, the resistance of piles, penetrating soft material, which depend solely upon skin friction, is materially increased after a period of rest. This period may be as short as fifteen minutes, and rarely exceeds twelve hours.

(19) In tidal waters the resistance of a pile driven at low tide is increased at high tide on account of the extra compression of the soil.

(20) Where a pile penetrates muck or a soft yielding material and bears upon a hard stratum at its foot, its strength should be determined as a column or beam; omitting the resistance, if any, due to skin friction.

(21) Unless the record of previous experience at the same site is available, the approximate bearing power may be obtained by loading test piles. The results of loading test piles should be used with caution, unless their condition is fairly comparable with that of the piles in the proposed foundation.

(22) In case the piles in a foundation are expected to act as columns the results of loading test piles should not be depended upon unless they are sufficient in number to insure their action in a similar manner, and they are stayed against lateral motion.

(23) Before testing the penetration of a pile in soft material where its bearing power depends principally, or wholly, upon skin friction, the pile should be allowed to rest for 24 hours after driving.

(24) Where the resistance of piles depends mainly upon skin friction it is possible to diminish the combined strength, or bearing capacity, of a group of piles by driving additional piles within the same area.

(25) Where there is a hard stratum overlying softer material through which the piles are to pass to a firm bearing below, the upper stratum should be removed by dredging or otherwise, provided it would injure the piles to drive through the stratum. The material removed may be replaced if it is needed to provide lateral resistance.

(26) Timber piles may be advantageously pointed, in some cases, to a 4-in. or 6-in. square at the end.

(27) Piles should not be pointed when driven into soft material.

(28) Shoes should be provided for piles when the driving is very hard, especially in riprap or shale, and should be so constructed as to form an integral part of the pile.

(29) The use of a cap is advantageous in distributing the impact of the hammer more uniformly over the head of the pile, as well as to hold it in position during driving.

(30) The specification relating to the penetration of a pile should be adapted to the soil which the pile is to penetrate.

(31) It is far more important that a proper length of pile should be put in place without injury than that its penetration should be a specified distance under a given blow, or series of blows.

SPECIFICATIONS FOR TIMBER PILES.*

Railroad Heart Grade.

1. This grade includes white, burr, and post oak, longleaf pine, Douglas fir, tamarack, Eastern white and red cedar, chestnut, Western cedar, redwood and cypress.

2. Piles shall be cut from sound trees; shall be close grained and solid, free from defects, such as injurious ring shakes, large and unsound or loose knots, decay or other defects, which may materially impair their strength or durability. In Eastern red or white cedar a small amount of heart rot at the butt, which does not materially injure the strength of the pile, will be allowed.

*3. Piles must be butt cut above the ground swell and have a uniform taper from butt to tip. Short bends will not be allowed. A line drawn from the center of the butt to the center of the tip shall lie within the body of the pile.

4. Unless otherwise allowed, piles must be cut when sap is down. Piles must be peeled soon after cutting. All knots shall be trimmed close to the body of the pile.

5. For round piles the minimum diameter at the tip shall be nine (9) in. for lengths not exceeding thirty (30) ft.; eight (8) in. for lengths over thirty (30) ft. but not exceeding fifty (50) ft., and seven (7) in. for lengths over fifty (50) ft. The minimum diameter at one-quarter of the length from the butt shall be twelve (12) in. and the maximum diameter at the butt twenty (20) in.

6. For square piles the minimum width of any side of the tip shall be nine (9) in. for lengths not exceeding thirty (30) ft.; eight (8) in. for lengths over thirty (30) ft. but not exceeding fifty (50) ft., and seven (7) in. for lengths over fifty (50) ft. The minimum width of any side at one-quarter of the length from the butt shall be twelve (12) in.

7. Square piles shall show at least eighty (80) per cent heart on each side at any cross-section of the stick, and all round piles shall show at least ten and one-half ($10\frac{1}{2}$) in. diameter of heart at the butt.

Railroad Falsework Grade.

8. This grade includes red and all other oaks not included in R. R. Heart grade, sycamore, sweet, black and tupelo gum, maple, elm, hickory, Norway pine, or any sound timber that will stand driving.

9. The requirements for size of tip and butt, taper and lateral curvature are the same as for R. R. Heart grade.

10. Unless otherwise specified piles need not be peeled.

11. No limits are specified as to the diameter or proportion of heart.

12. Piles which meet the requirements of R. R. Heart grade except the proportion of heart specified will be classed as R. R. Falsework grade.

GUARD RAILS AND GUARD TIMBERS.—In 1912 the American Railway Engineering Association made an investigation of the use of guard rails and guard timbers for timber trestles and bridges and adopted the following report based on replies from 61 railroads.

1. It is recommended as good practice to use guard timbers on all open-floor bridges, and same shall be so constructed as to properly space the ties and hold them securely in their places.

2. It is recommended as good practice to use guard rails to extend beyond the end of the bridges for such a distance as required by local conditions, but that this length in any case be not less than fifty feet; that guard rails be fully spiked to every tie and spliced at every joint, the guard rail to be some form of metal guard rail.

3. It is recommended that the guard timber and guard rail be so spaced in reference to the track rail that a derailed truck will strike the guard rail without striking the guard timber.

4. The height of the guard rail to be not over one inch less in height than the running (track) rail.

TIMBER TRESTLES.—The details of the design of timber trestles depends upon the loading, the details of the floor system, the available timber and upon the designer. The length of panels varies from 12 ft. to 16 ft., with 14 ft. as a fair average panel length.

Pile Trestles.—The details of the standard pile trestle with open floor of the N. Y., N. H. & H. R. R. are given in Fig. 1. The number and arrangement of the piles in the bents are shown. The bents are 12 ft. center to center. The stringers are 24 ft. long and are placed to span two panels and to break joints. The tops of the caps are covered with No. 20 flat galvanized iron to protect the trestle from fire. The details of washers, packing blocks, drift bolts, etc., are shown on the plans.

* Adopted, Am. Ry. Eng. Assoc., Vol. 10, 1909.

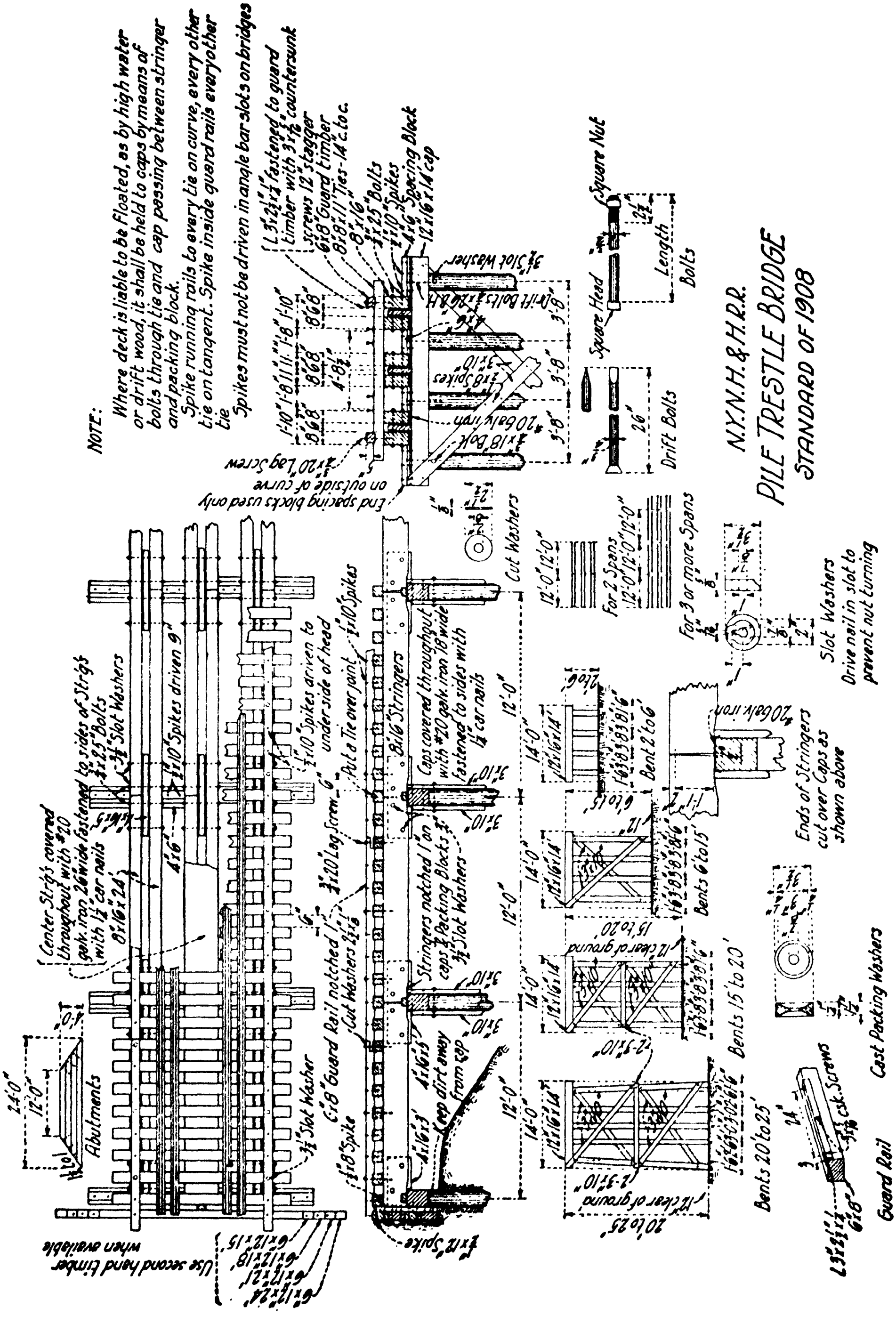


FIG. 1. PLANS OF A PILE TRESTLE.

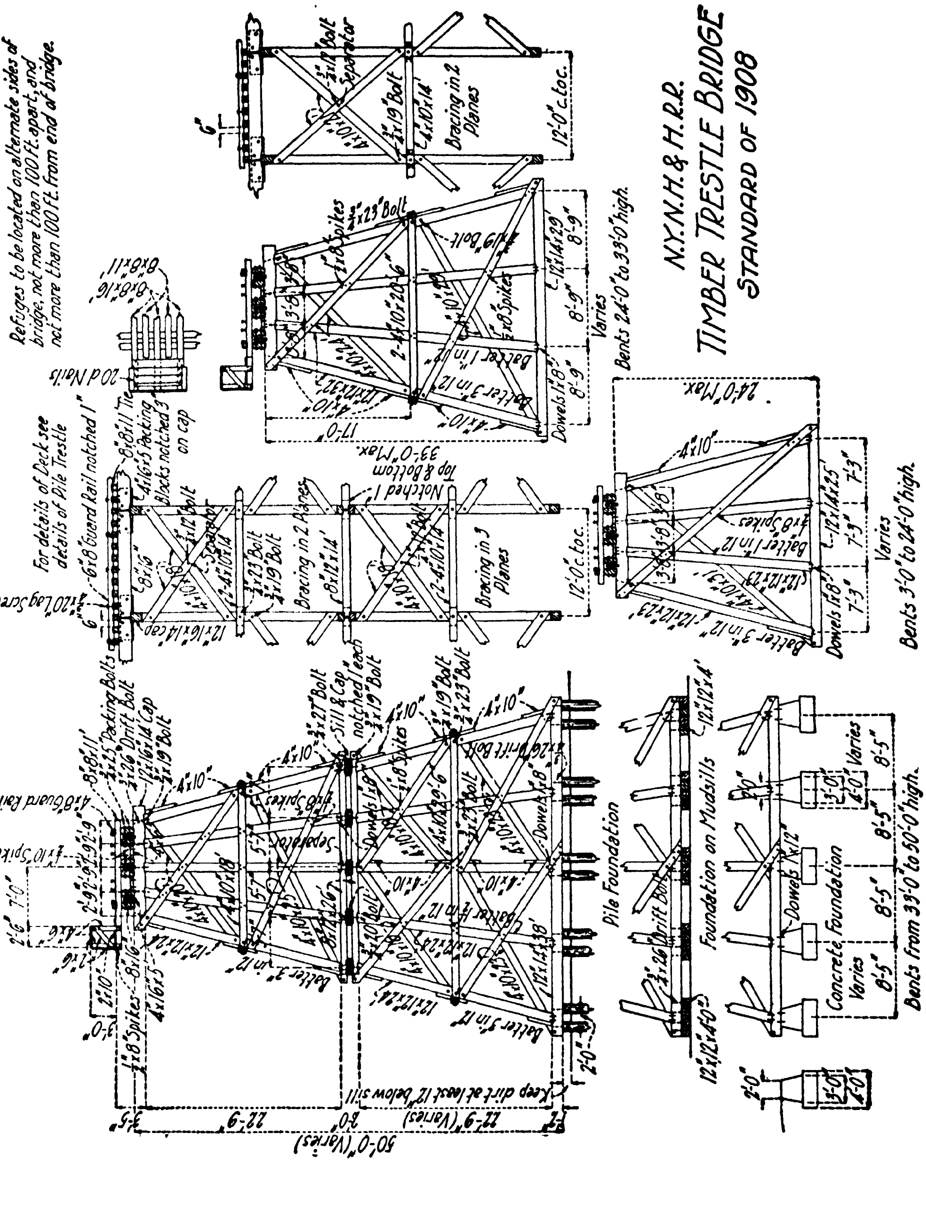
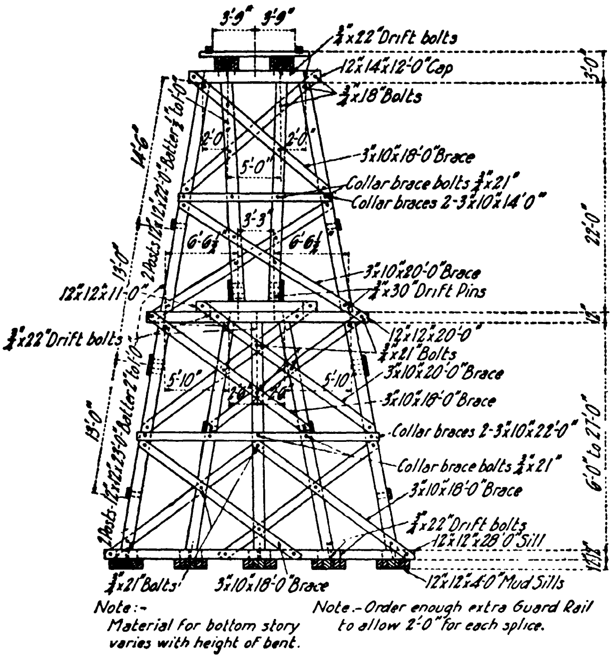


FIG. 2. PLANS OF A FRAME TRESTLE.



	No.	2 STORY BENT 50'-0" HIGH — DESCRIPTION	BD. FT.
TIMBER	2	Pcs. 3"x10"x14'-0" Braces	70
	8	" 3"x10"x18'-0" "	360
	4	" 3"x10"x20'-0" "	200
	2	" 3"x10"x22'-0" "	110
	1	" 12"x14"x12'-0" Cap	168
	1	" 12"x12"x12'-0" "	144
	1	" 12"x12"x20'-0" "	240
	2	" 12"x12"x22'-0" Posts	528
	7	" 12"x12"x23'-0" "	1932
	1	" 12"x12"x28'-0" Sill	336
	15	" 12"x12"x4'-0" Mud Sills	720
IRON			WT.
	4	¾" Square Drift Pins 30" Long	19.0
	29	¾" " " Bolts 22" "	104.4
	19	¾"x21" Bolts	56.8
	36	¾"x18" "	93.6
	55	Plain Washers	66.0
	55	Slotted "	66.0
	55	10 Penny Wire Nails	0.7
		FOR 1 PANEL OF FLOOR	
TIMBER	No.	DESCRIPTION	BD. FT.
	3	Pcs. 7"x16"x28'-0" Stringers	784
	10	" 6"x8"x10'-0" Bridge Ties	400
	2	" 8"x8"x10'-0" Lining	107
	1	" 6"x8"x28'-0" Guard Rail	112
IRON			WT.
	6	¾" Square Drift Bolts 22" Long	21.6
	8	¾"x28" Bolts	30.8
	6	¾"x12" " Northern Lines only	11.3
	6	¾"x14" " Southern " "	12.9
	16	Cast Iron Packing Washers	48.0
	6	Inset Washers Northern Lines only	9.0
	8	Plain " " " "	9.6
	14	" " Southern " "	16.8
	14	Slotted "	16.8
	14	10 Penny Wire Nails	0.2

FIG. 3. PLANS OF TIMBER FRAME TRESTLE. ILLINOIS CENTRAL RAILROAD.

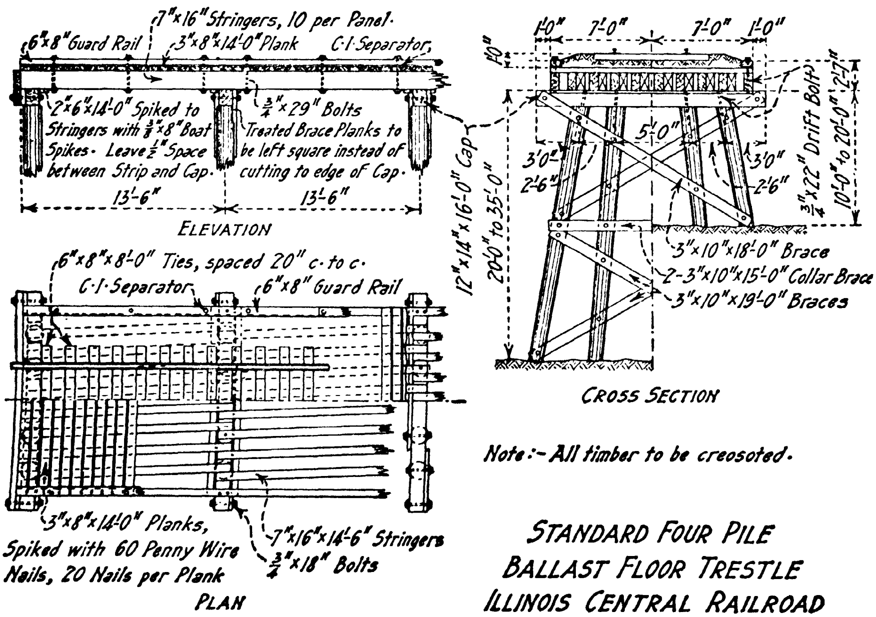


FIG. 4. PLANS OF TIMBER TRESTLE WITH BALLASTED DECK.

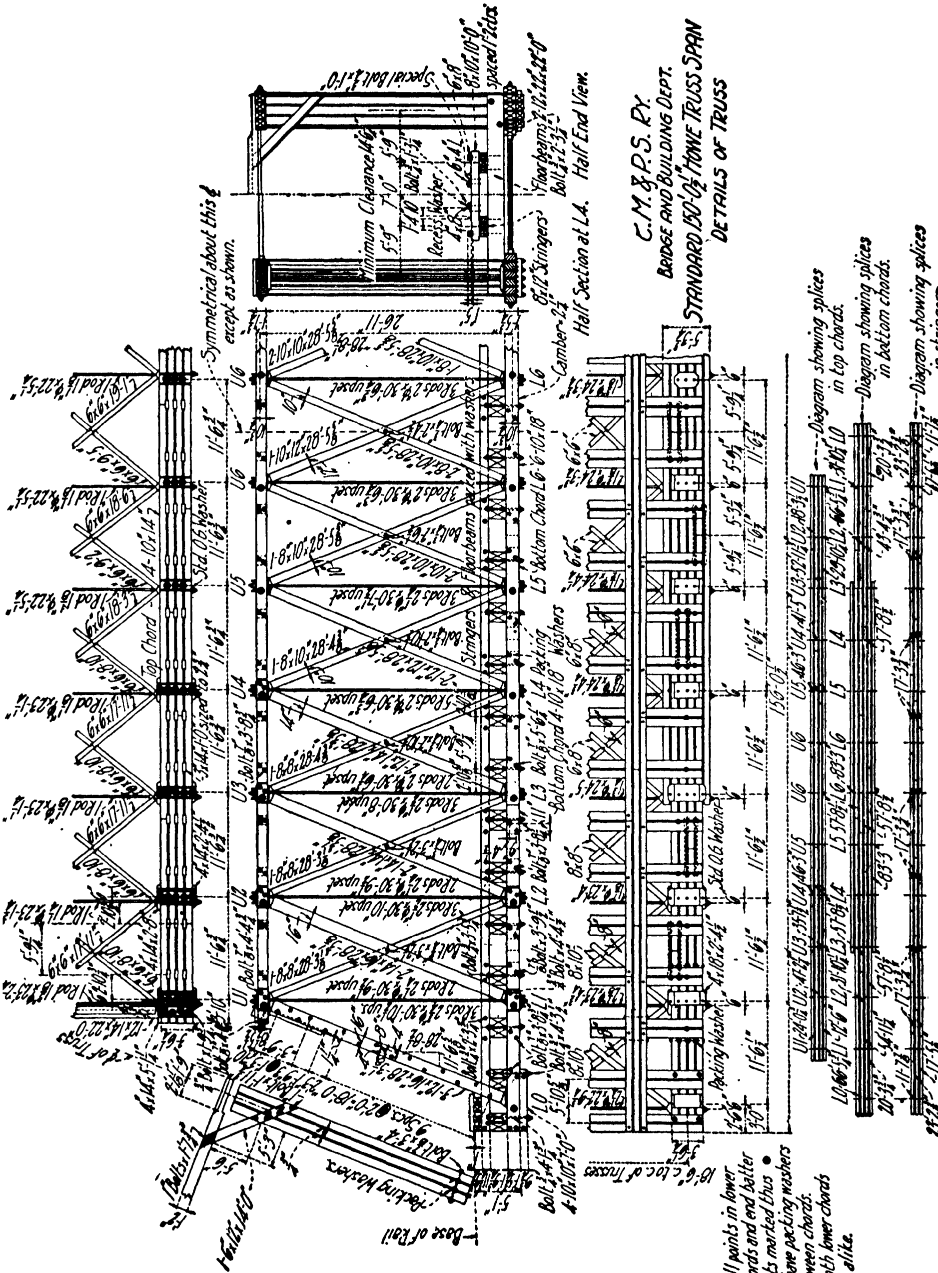

C.M.&P.S. RY.
BRIDGE AND BUILDING DEPT.
STANDARD 150'-0½" HOWE TRUSS SPAN
DETAILS OF TRUSS
Half Section at L4. Half End View.
Symmetrical about this ℄ except as shown.
Top Chord
Diagram showing splices in top chords.
Diagram showing splices in bottom chords.
Diagram showing splices in stringers.
All joints in lower chords and end batter posts marked thus ● to have packing washers between chords. Both lower chords are alike.

FIG. 5.

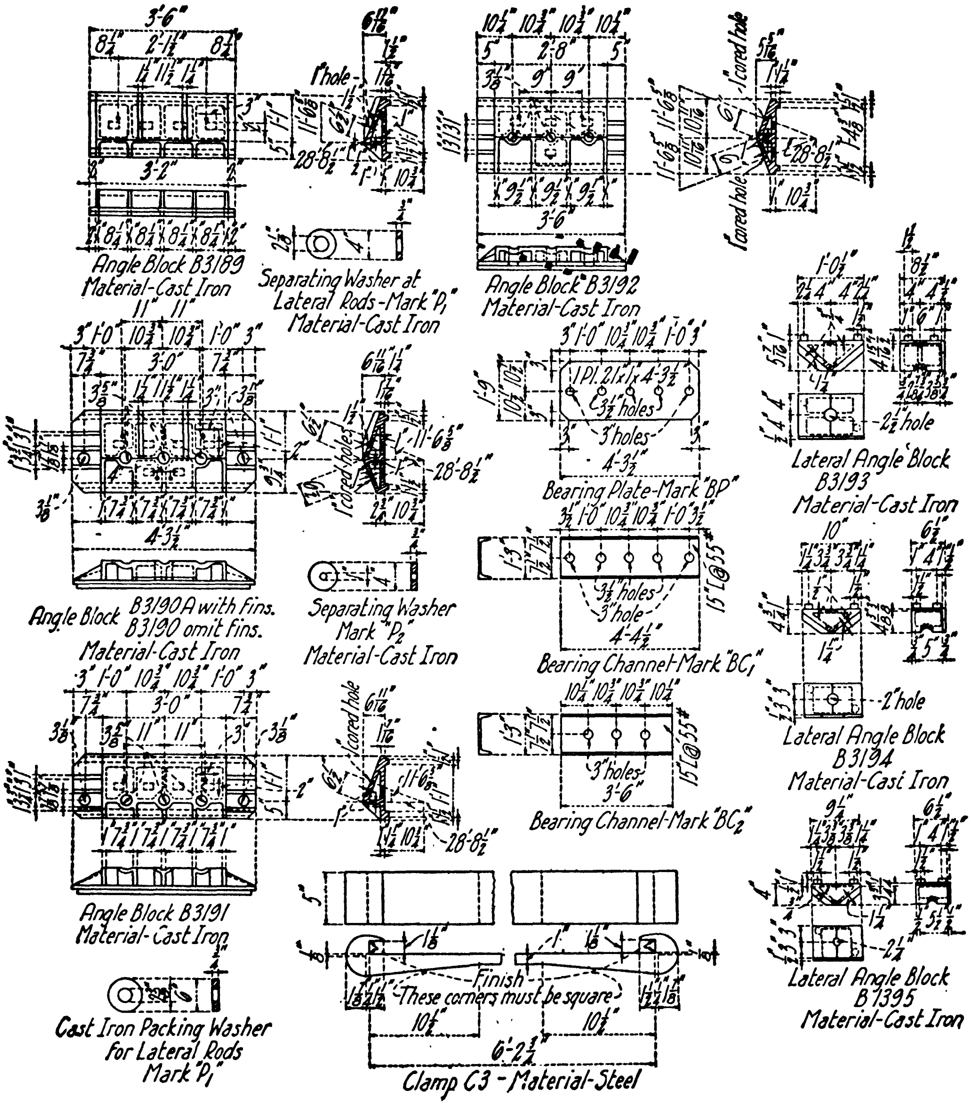


FIG. 6. IRON DETAILS FOR 150 FT. SPAN, HOWE TRUSS SPAN.
C. M. & P. S. RY.

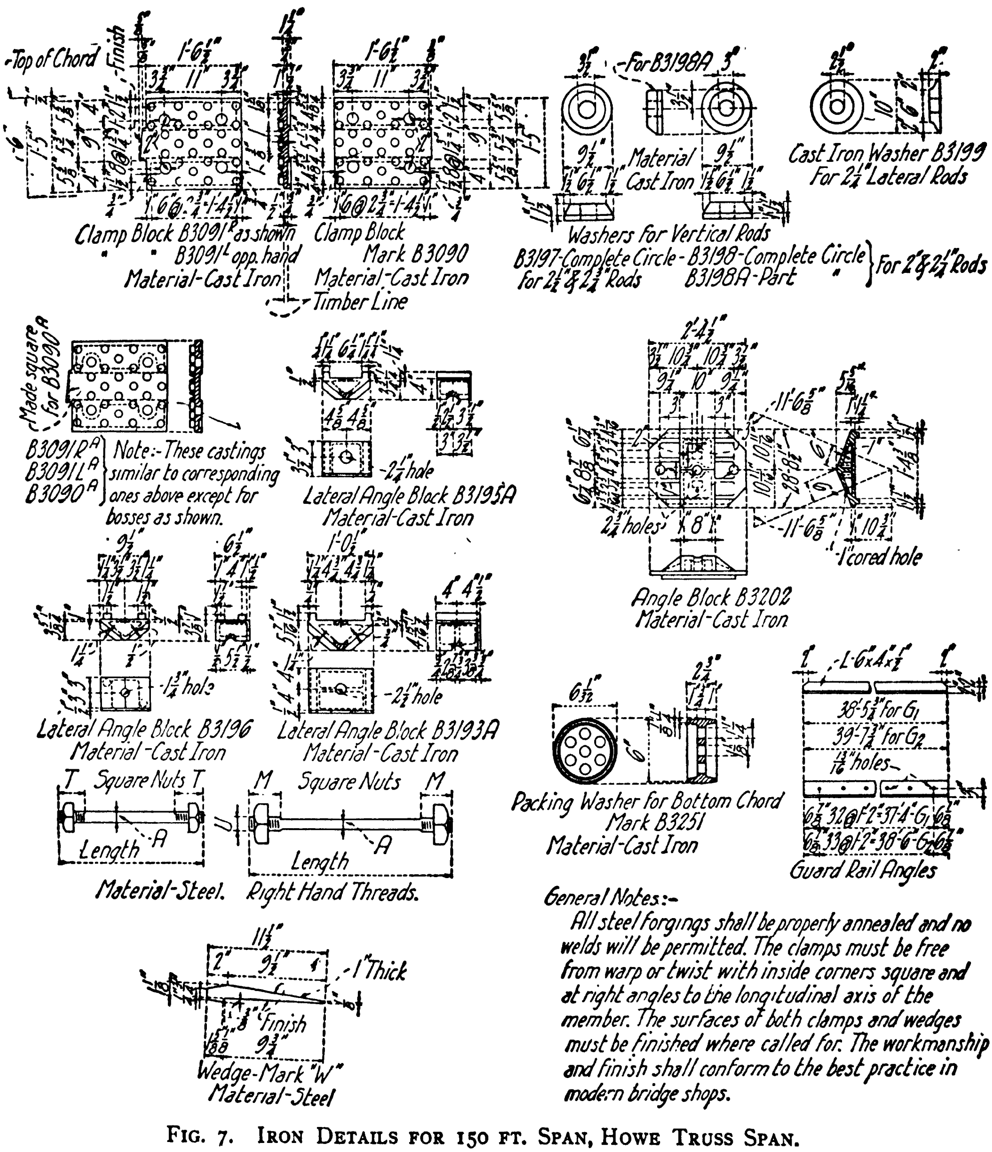


FIG. 7. IRON DETAILS FOR 150 FT. SPAN, HOWE TRUSS SPAN.
C. M. & P S RY

Frame Trestles.—The details of the standard frame trestle with open floor ot the N. Y., N. H. & H. R. R. are given in Fig. 2. The bents are spaced 12 ft. center to center. The floor system is the same as for pile trestles. The frame trestle may be supported on a pile foundation, upon timber sub-sills (mudsills) or on concrete pedestals. Timber sub-sills soon decay and should be used only for temporary trestles. Other data and details are shown on the plans.

The plans of a standard frame trestle designed and built by the Illinois Central Railroad are given in Fig. 3. The bents are spaced 14 ft. centers, while the stringers are 28 ft. long and cover two panels. The details of the track and the guard rails are not shown. A complete bill of timber and iron for one bent and one panel of the floor are given in Fig. 3. The standard frame trestle may be carried on mudsills (sub-sills) as shown in Fig. 3, or on piles or concrete pedestals as shown in Fig. 2.

Detail plans of a pile trestle with ballasted deck are given in Fig. 4.

TIMBER HOWE TRUSSES.—Plans of a standard 150 ft. span Howe truss designed and erected by the C. M. & P. S. Ry. are shown in Fig. 5, Fig. 6, and Fig. 7. This bridge was designed for Cooper's E 55 Loading, with the allowable unit stresses as given in the American Railway Engineering Association Specifications for Timber Bridges and Trestles. The bill of lumber is given in Table I; the bill of castings and bolts is given in Table II; the bill of upset vertical rods is given in Table III, and the bill of lateral rods is given in Table IV. The following additional specifications were given on the plans.

TABLE I.

BILL OF TIMBER FOR ONE 150 FT. HOWE TRUSS SPAN.

No. of Pcs.	Size, In.	Length, Ft.-In.	Location.	No. of Pcs.	Size, In.	Length, Ft.-In.	Location.
2	10 × 14	12-6	Top Chord.	8	8 × 8	28-3⅛	Diag. Posts.
2	" " "	18-3½	" "	2	12 × 14	22-0	Portal.
2	" " "	24-0¾	" "	4	6 × 12	14-0	"
2	" " "	29-10¼	" "	2	8 × 10	9-0	Bott. Laterals.
2	" " "	35-7½	" "	2	" " "	8-7	" "
2	" " "	41-5	" "	2	" " "	18-0	" "
12	" " "	46-3	" "	2	" " "	17-9	" "
2	" " "	47-2¼	" "	4	" " "	8-8	" "
2	" " "	52-11¾	" "	2	8 × 8	17-4	" "
16	4 × 14	2-4½	" "	2	" " "	8-1	" "
12	" " "	2-8	" "	2	" " "	8-9	" "
4	" " "	5-1¼	" "	4	6 × 8	17-0	" "
132	3 × 14	1-0	" "	8	" " "	8-5	" "
4	10 × 18	20-3¾	Bott. Chord.	4	6 × 6	8-5	" "
4	" " "	31-10¼	" "	2	" " "	17-1	" "
2	" " "	43-4¾	" "	1	" " "	17-8	" "
2	" " "	54-11¼	" "	2	" " "	8-9	" "
10	" " "	57-8½	" "	14	" " "	8-10	Top Laterals.
4	" " "	66-5¾	" "	6	" " "	17-11	" "
4	" " "	83-3	" "	4	" " "	9-2	" "
8	4 × 18	2-4½	" "	2	" " "	9-5	" "
8	" " "	2-8	" "	2	" " "	18-6	" "
16	10 × 10	7-0	Corbels.	2	" " "	9-3	" "
12	12 × 16	28-3⅛	Diag. Posts.	2	" " "	18-3	" "
8	14 × 16	" "	" "	1	" " "	19-1	" "
8	14 × 14	" "	" "	56	12 × 22	22-0	Floorbeams.
8	12 × 14	" "	" "	4	8 × 12	23-2⅜	Stringers.
8	12 × 12	28-5⅝	" "	42	" " "	17-3¾	"
2	10 × 12	" "	" "	4	" " "	11-7⅞	"
8	10 × 10	" "	" "	4	" " "	17-5⅛	"
8	8 × 10	" "	" "	134	8 × 10	10-0	Ties.
4	8 × 10	28-4⅜	" "	21	6 × 8	16-0	Guard Rail.
4	8 × 8	" "	" "	21	4 × 8	16-0	" "

Lengths given for Top and Bottom Laterals are longer than finished lengths.

TABLE II.

BILL OF CASTINGS, BOLTS, ETC. FOR ONE 150 FT. HOWE TRUSS SPAN.

No. of Pcs.	Description.	Mark.	No. of Pcs.	Description.	Mark.
4	Angle Blocks	B3189	18	Dowels $\frac{7}{8}$ in. × 0 ft.–9 in.	
20	" "	B3190	176	Dowels $\frac{7}{8}$ in. × 0 ft.–3 in.	
4	" "	B3191	275	Spikes 9 in. × $\frac{3}{8}$ in.	
16	" "	B3192	225	" 8 in. × $\frac{3}{8}$ in.	
2	" "	B3202	275	" 14 in. × $\frac{1}{2}$ in.	
12	Lateral Angle Blocks	B3193	115	Drift Bolts $\frac{3}{4}$ in. × 1 ft. 8 in.	
4	" " "	B3193A	24	Bolts 1 in. × 1 ft.–11$\frac{3}{4}$ in.	
12	" " "	B3194		Sq. H & N 2$\frac{1}{2}$ in. thd.	
12	" " "	B1395	24	Bolts 1 in. × 1 ft.–7$\frac{3}{4}$ in.	
8	" " "	B3196		Sq. H & N 2$\frac{1}{2}$ in. thd.	
30	Clamp Blocks	B3090	56	Bolts $\frac{7}{8}$ in. × 5 ft.–6$\frac{1}{4}$ in.	
6	" "	B3090A		Sq. H & N 2$\frac{1}{2}$ in. thd.	
15	" "	B3091R	32	Bolts $\frac{7}{8}$ in. × 4 ft.–4$\frac{3}{4}$ in.	
3	" "	B3091RA		Sq. H & N 2$\frac{1}{2}$ in. thd.	
15	" "	B3091L	8	Bolts $\frac{7}{8}$ in. × 4 ft.–$\frac{3}{4}$ in.	
3	" "	B3091LA		Sq. H & N 2$\frac{1}{2}$ in. thd.	
4	Washers for Lateral Rods	B 3199	142	Bolts $\frac{7}{8}$ in. × 3 ft.–8$\frac{3}{4}$ in.	
72	" " " "	B3197		Sq. H & N 2$\frac{1}{2}$ in. thd.	
72	" " " "	B3198	24	Bolts $\frac{7}{8}$ in. × 3 ft.–9$\frac{1}{2}$ in.	
64	" " " "	B3198A		Sq. H & N 2$\frac{1}{2}$ in. thd.	
4	O. G. Washers for 2$\frac{1}{8}$ in. Bolts		60	Bolts $\frac{7}{8}$ in. × 3 ft.–4 in.	
4	" " " 1$\frac{1}{4}$ " "			Sq. H & N 2$\frac{1}{2}$ in. thd.	
4	" " " 1$\frac{7}{8}$ " "		16	Bolts $\frac{3}{4}$ in. × 1 ft.–9$\frac{3}{4}$ in.	
4	" " " 1$\frac{3}{4}$ " "			Sq. H & N 2$\frac{1}{2}$ in. thd.	
4	" " " 1$\frac{5}{8}$ " "		56	Bolts $\frac{3}{4}$ in. × 2 ft.–3$\frac{1}{4}$ in.	
4	" " " 1$\frac{1}{2}$ " "			Sq. H & N 2$\frac{1}{2}$ in. thd.	
8	" " " 1$\frac{3}{8}$ " "		72	Bolts $\frac{3}{4}$ in. × 2 ft.–3$\frac{3}{4}$ in.	
16	" " " 1$\frac{1}{8}$ " "			Sq. H & N 2$\frac{1}{2}$ in. thd.	
48	" " " 1 " "		2	Bolts $\frac{3}{4}$ in. × 2 ft.–4$\frac{1}{2}$ in.	
322	" " " $\frac{7}{8}$ " "			Sq. H & N 2$\frac{1}{2}$ in. thd.	
246	" " " $\frac{3}{4}$ " "		4	Bolts $\frac{3}{4}$ in. × 2 ft.–6$\frac{1}{2}$ in.	
48	Slot Washers for 1 in. Bolts			Sq. H & N 2$\frac{1}{2}$ in. thd.	
322	" " " $\frac{7}{8}$ " "		8	Bolts $\frac{3}{4}$ in. × 2 ft.–10$\frac{1}{2}$ in.	
410	" " " $\frac{3}{4}$ " "			Sq. H & N 2$\frac{1}{2}$ in. thd.	
4	6 in. × 4 in. × $\frac{1}{2}$ in. × 38 ft.–		8	Bolts $\frac{3}{4}$ in. × 3 ft.–2$\frac{1}{2}$ in.	
	5$\frac{3}{4}$ in. Guard Angles	G_1		Sq. H & N 2$\frac{1}{2}$ in. thd.	
4	6 in. × 4 in. × $\frac{1}{2}$ in. ×		48	Bolts $\frac{3}{4}$ in. × 3 ft.–5$\frac{3}{4}$ in.	
	39 ft.–7$\frac{3}{4}$ in. Guard Angles	G_2		Sq. H & N 2$\frac{1}{2}$ in. thd.	
424	Packing Washers	B3251	8	Bolts $\frac{3}{4}$ in. × 4 ft.–1$\frac{3}{4}$ in.	
36	" "	P_1		Sq. H & N 2$\frac{1}{2}$ in. thd.	
152	" "	P	8	Bolts $\frac{3}{4}$ in. × 4 ft.–3$\frac{1}{2}$ in.	
416	" "	P_2		Sq. H & N 2$\frac{1}{2}$ in. thd.	
36	Clamps	C_3	16	Bolts $\frac{3}{4}$ in. × 4 ft.–4$\frac{1}{2}$ in.	
36	Wedges	W		Sq. H & N 2$\frac{1}{2}$ in. thd.	
16	Bearing Plates	BP	64	Bolts $\frac{3}{4}$ in. × 1 ft.–3$\frac{1}{4}$ in.	
32	Bearing Channels	BC1		Sq. H & N 2$\frac{1}{2}$ in. thd.	
16	" "	BC2	64	Recess Washers	
4	Angle Blocks	B3190A	100	Special Bolts $\frac{3}{4}$ in. × 1 ft.	B3195A
6	Dowels $\frac{7}{8}$ in. × 0 ft.–11 in.		4	Lateral Angle Blocks	B3192A
	steel		2	Angle Blocks	

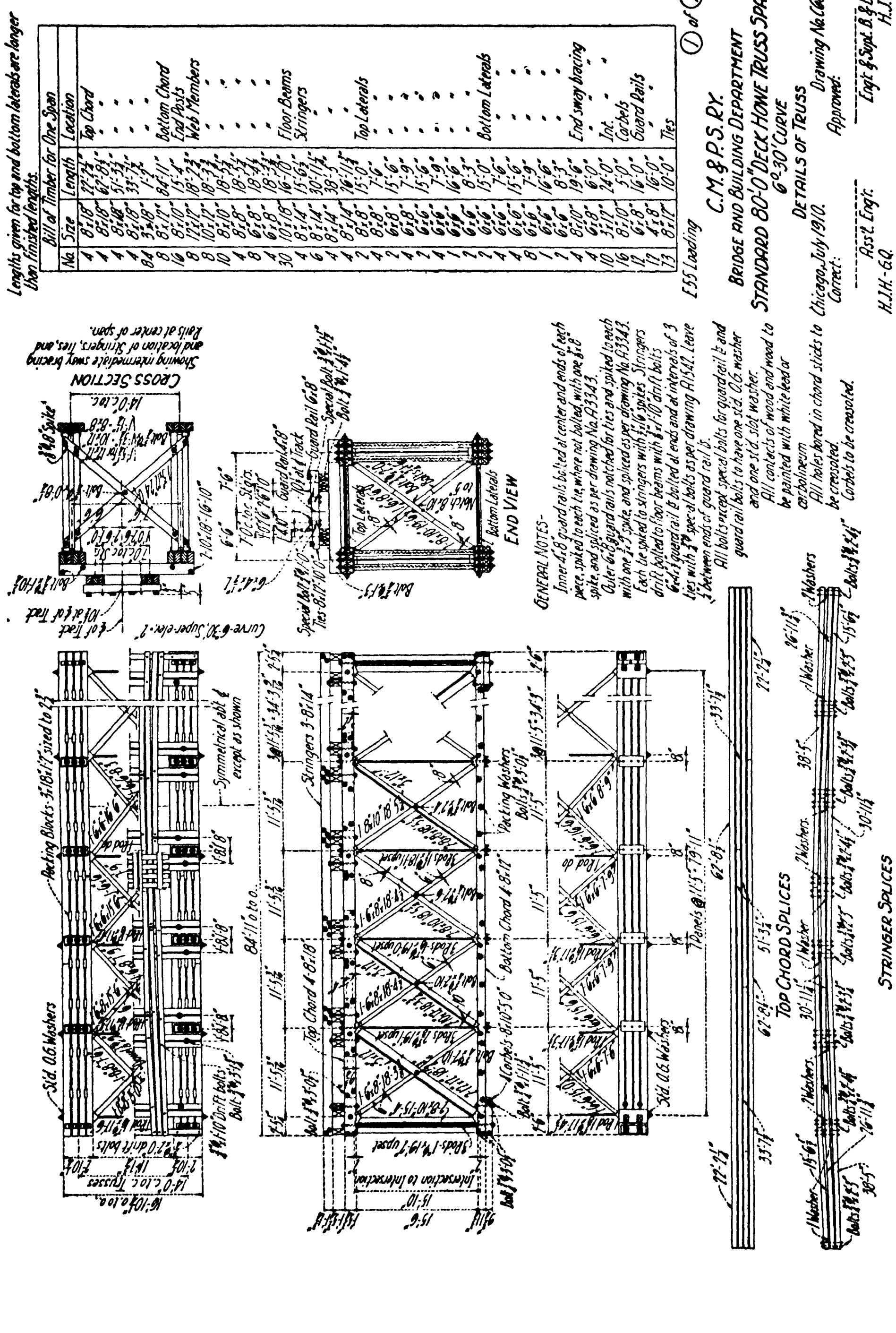

Lengths given for top and bottom laterals are longer than finished lengths.
Bill of Timber for One Span
No. Size Length Location
Top Chord
Bottom Chord
End Posts
Web Members
Floor Beams
Stringers
Top Laterals
Bottom Laterals
End sway bracing
Int.
Corbels
Guard Rails
Ties
E55 Loading
1 of 2
C.M.&P.S.RY.
BRIDGE AND BUILDING DEPARTMENT
STANDARD 80'-0" DECK HOWE TRUSS SPAN
6°30' CURVE
DETAILS OF TRUSS
Chicago, July 1910.
Drawing No. C6833
Correct:
Approved:
Ass't. Engr.
Engr. & Supt. B. & B.
H.J.H.-G.Q.
H.J.C.
CROSS SECTION
Showing intermediate sway bracing and location of Stringers, Ties, and Rails at center of span.
END VIEW
Top Laterals
Bottom Laterals
GENERAL NOTES-
All bolts except special bolts for guard rail and guard rail bolts to have one std. O.G. washer and one std. slot washer.
All contacts of wood and wood to be painted with white lead
All holes bored in chord sticks to be creosoted.
Corbels to be creosoted.
Packing Blocks
Std. O.G. Washers
Symmetrical abt. ℄ except as shown
Stringers 3-8"x14"
Top Chord 4-8"x18"
Bottom Chord 4-8"x12"
Corbels
Packing Washers
Intersection to Intersection
Panels @ 11'-5"
TOP CHORD SPLICES
STRINGER SPLICES
Washers

FIG. 8.

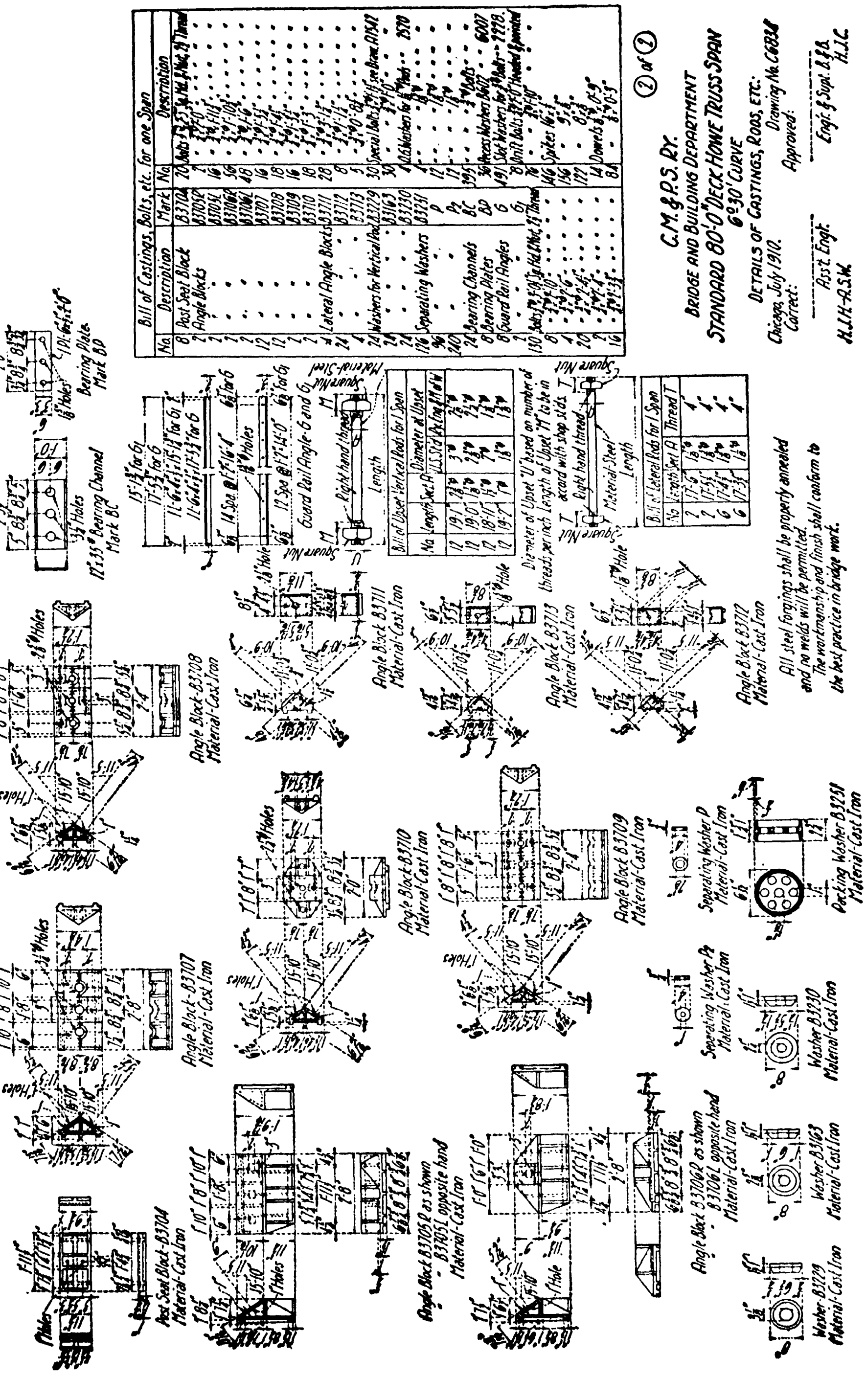

C.M.&P.S. RY.
BRIDGE AND BUILDING DEPARTMENT
STANDARD 80'-0" DECK HOWE TRUSS SPAN
6°30' CURVE
DETAILS OF CASTINGS, RODS, ETC.
Chicago, July 1910.
Correct:
Approved:
Asst. Engr.
Engr. & Supt. B.&B.
Bill of Castings, Bolts, etc. for one Span
All steel forgings shall be properly annealed and no welds will be permitted.
The workmanship and finish shall conform to the best practice in bridge work.
Post Seat Block-B3704 Material-Cast Iron
Angle Block-B3707 Material-Cast Iron
Angle Block-B3708 Material-Cast Iron
Angle Block-B3709 Material-Cast Iron
Angle Block-B3710 Material-Cast Iron
Angle Block-B3711 Material-Cast Iron
Angle Block-B3712 Material-Cast Iron
Angle Block-B3713 Material-Cast Iron
Bearing Plate Mark BD
Bearing Channel Mark BC
Guard Rail Angle-6 and G1
Separating Washer P Material-Cast Iron
Separating Washer P2 Material-Cast Iron
Packing Washer-B3251 Material-Cast Iron
Washer-B3230 Material-Cast Iron
Washer-B3163 Material-Cast Iron
Washer-B3229 Material-Cast Iron

FIG. 9.

"Outer 6 in. × 8 in. Guard Rails are notched for ties, spiked to each tie with one 9 in. × ⅝ in. spikes. Each tie to be spiked to stringers with ⅝ in. × 14 in. spikes. Stringers drift-bolted to floorbeams with ¾ in. × 18 in. drift bolts. All ¾ in., ⅞ in. and 1 in. bolts to be provided with one O. G. and one slot washer. All contacts of wood and wood to be painted with white lead. Corbels to be creosoted. All holes bored in chord sticks to be creosoted. Inner 4 in. × 8 in. Guard Rails bolted at center and ends of each piece, spiked to each tie not bolted, with one 8 in. × ⅝ in. spike and spliced. The 6 in. × 4 in. × ½ in. guard rail is bolted at ends and at intervals of not over 3 ties with ¾ in. special bolts. Leave ¼ in. opening between ends of Guard Rail angles."

The detail plans of a timber Howe truss railway bridge with an 80 ft. span are given in Fig. 8 and Fig. 9. This bridge was designed for Cooper's E 55 loading for the allowable stresses given in the specifications of the American Railway Engineering Association. The details and a bill of materials are given on the plans.

TABLE III.

BILL OF UPSET VERTICAL RODS FOR ONE 150 FT. HOWE TRUSS SPAN.

No. of Pcs.	Length, Ft.-In.	Section "A" Diam., In.	Diameter of Upsets. U. S. Std., In.	Diameter of Upsets. Ry. Eng. & M. of W., In.
12	30-10½	2¾	3⅜	3¼
12	30-10	2½	3⅛	3
12	30- 8	2½	3⅛	3
16	30- 9½	2¼	2⅞	2¾
12	30- 7½	2¼	2⅞	2¾
40	30- 6¾	2	2½	2⅜

Diameter of Upset "U" based on number of threads per inch.
Length of upsets "M" to be in accord with shop standards.

TABLE IV.

BILL OF LATERAL RODS FOR ONE 150 FT. HOWE TRUSS SPAN.

No. of Pcs.	Length, Ft.-In.	Diameter of Rod "A," In.	Length of Thread "T," In.
2	22-9¾	2¼	5
2	23-4¾	2⅛	4½
2	23-4	1⅞	4½
2	24-5	1¾	4½
2	24-4¾	1⅝	4½
2	24-4¼	1½	4½
2	24-3¾	1⅜	4
2	23-2¼	1⅜	4
2	23-1¾	1¼	4
4	23-1¼	1⅛	4
4	22-5¼	1⅛	4

HIGHWAY TIMBER TRESTLES AND BRIDGES.—Details of a highway crossing of the Illinois Central Railroad are given in Fig. 10 and Fig. 11.

A combination timber and iron bridge is shown in Fig. 12; while a short span timber highway bridge is given in Fig. 13.

A timber truss bridge designed in 1920 by the Iowa Highway Commission is shown in Fig. 14.

For additional details of timber highway bridges, see the author's "The Design of Highway Bridges of Steel, Timber and Concrete."

SPECIFICATIONS FOR WORKMANSHIP FOR PILE AND FRAME TRESTLES TO BE BUILT UNDER CONTRACT.*

1. **Site.**—The trestle to be built under these specifications is located on the line of .... Railroad at .... County of .... State of ....

2. **General Description.**—The work to be done under these specifications covers the driving, framing and erection of a .... track wooden trestle about .... ft. long and an average of .... ft. high.

GENERAL CLAUSES.

3. The contractor shall furnish all necessary labor, tools, machinery, supplies, temporary staging and outfit required. He shall build the complete trestle ready for the track rails, in a workmanlike manner, in strict accordance with the plans and the true intent of these specifications, to the satisfaction and acceptance of the engineer of the railroad company.

4. The workmanship shall be of the best quality in each class of work. Details, fastenings and connections shall be of the best method of construction in general use on first class work.

* Adopted by American Railway Engineering Association.

5. Holes shall be bored for all bolts. The depth of the hole and the diameter of the auger to be specified by the engineer.

6. Framing shall be accurately fitted; no blocking or shimming will be allowed in making joints. Timbers shall be cut off with the saw; no axe to be used.

7. Joints and points of bearing, for which no fastening is shown on the plans, shall be fastened as specified by the engineer.

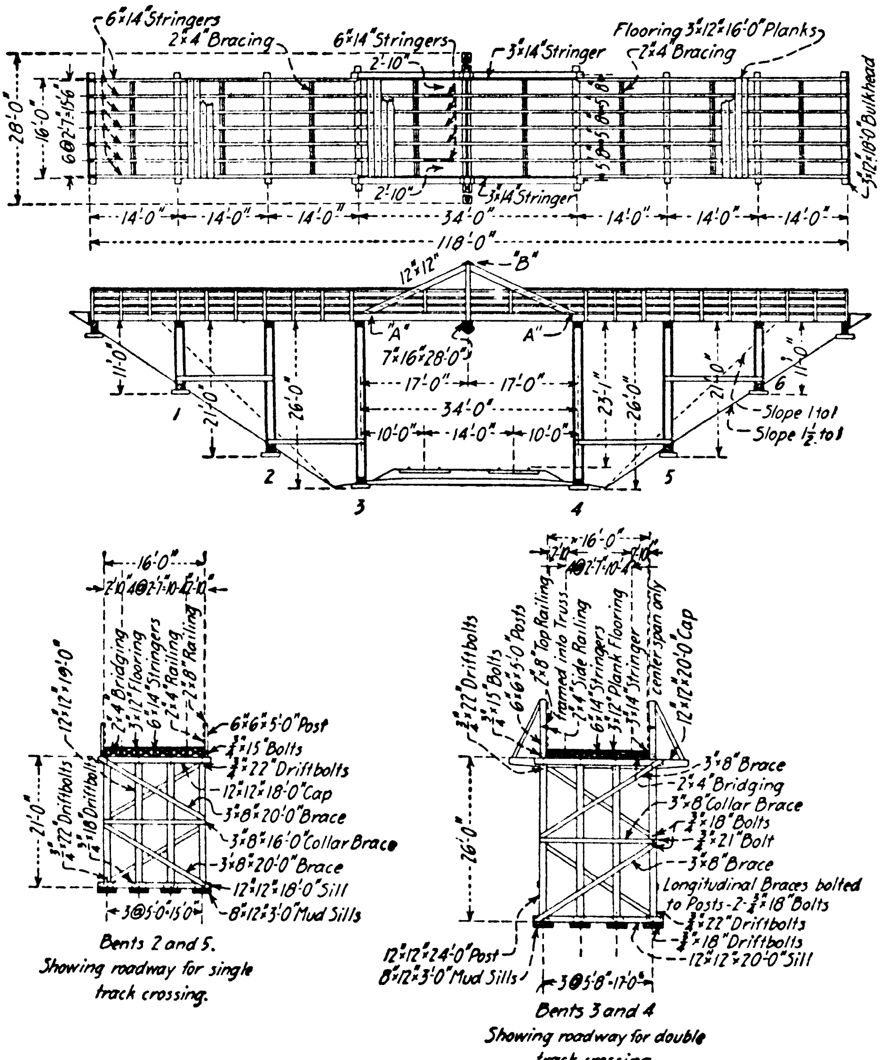


FIG. 10. HIGHWAY CROSSING. ILLINOIS CENTRAL RAILROAD.

8. The engineer or his authorized agents shall have full power to cause any inferior work to be condemned, and taken down or altered, at the expense of the contractor. Any material destroyed by the contractor on account of inferior workmanship or carelessness of his men is to be replaced by the contractor at his own expense.

9. Figures shown on the plans shall govern in preference to scale measurements; if any discrepancies should arise or irregularities be discovered in the plans, the contractor shall call on the engineer for instructions. These specifications and the plans are intended to co-operate, and if any question arises as to the proper interpretation of the plans or specifications, it shall be referred to the engineer for a ruling.

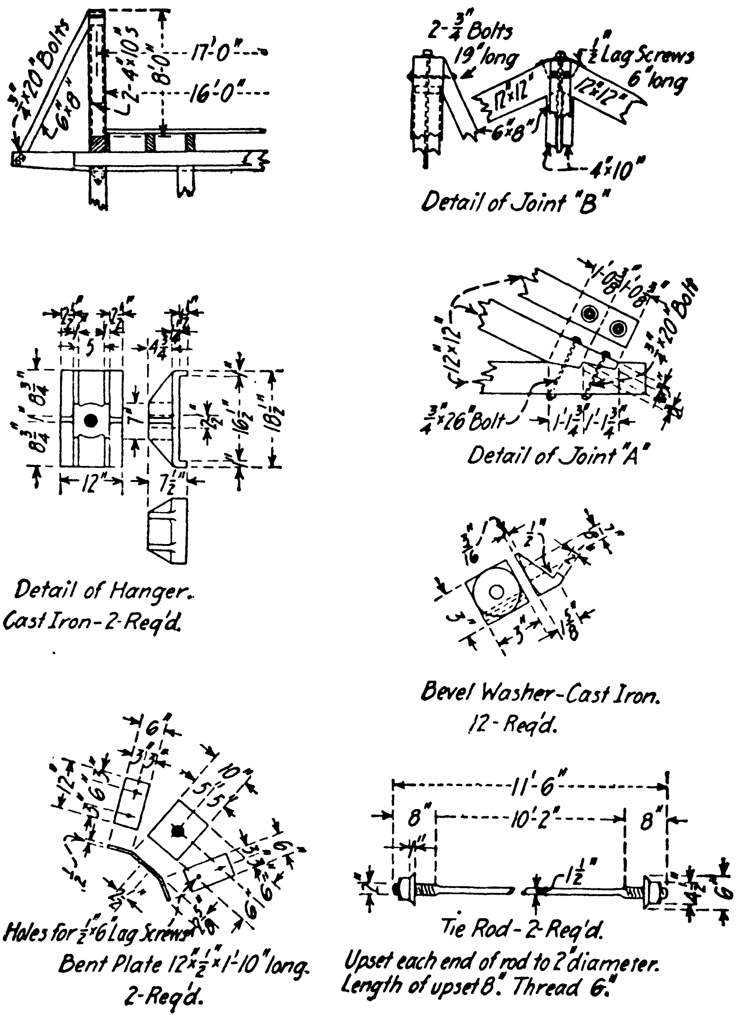


FIG. 11. DETAILS OF HIGHWAY CROSSING. ILLINOIS CENTRAL RAILROAD.

10. The contractor shall, when required by the engineer furnish a satisfactory watchman to guard the work.

11. On the completion of the work, all refuse material and rubbish that may have accumulated on top or under and near the trestle, by reason of its construction, shall be removed by the contractor.

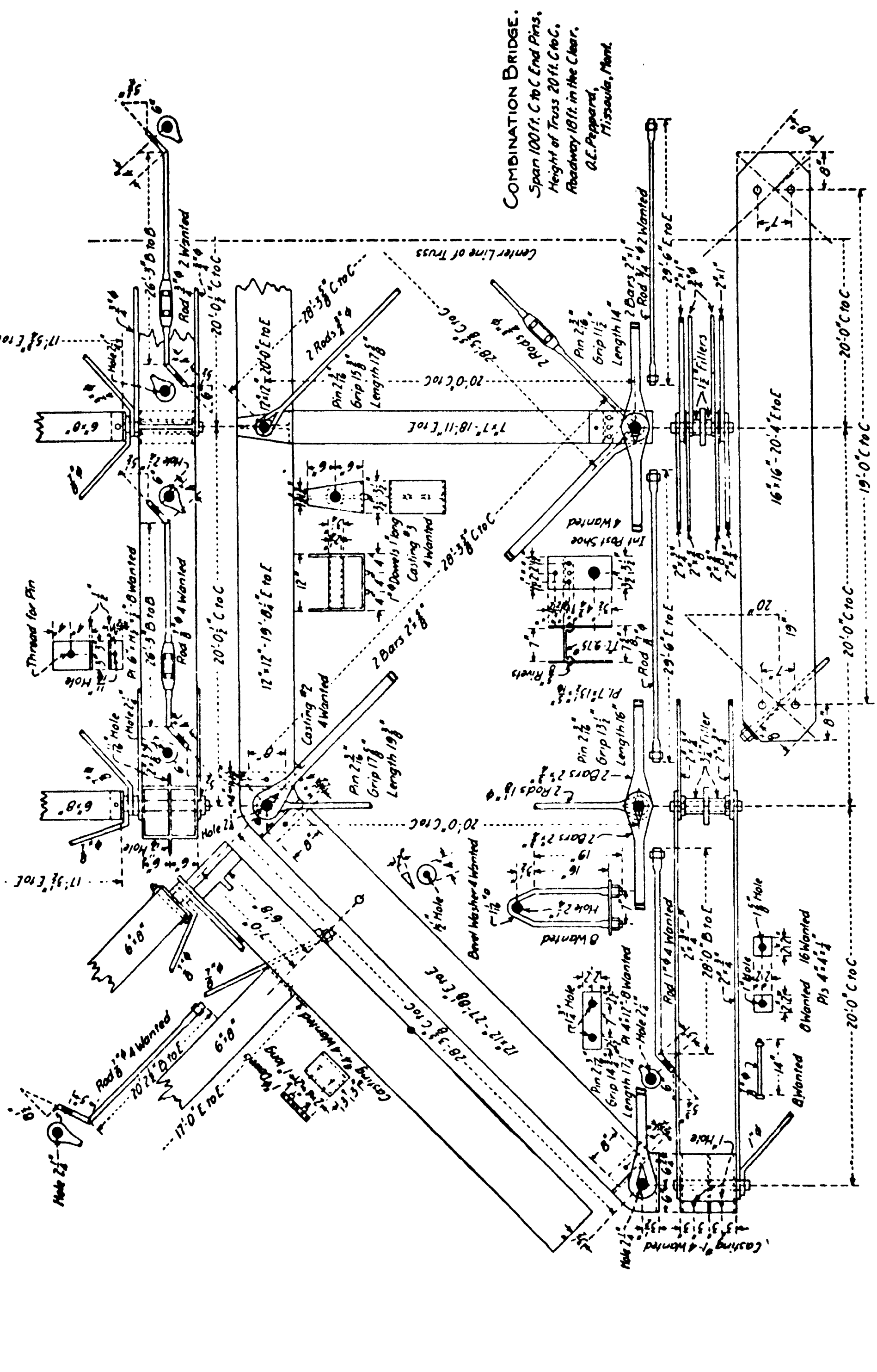

Combination Bridge.
Span 100 ft. C to C End Pins,
Height of Truss 20 ft. C to C,
Roadway 18 ft. in the Clear.
O.E. Peppard,
Missoula, Mont.

FIG. 12.

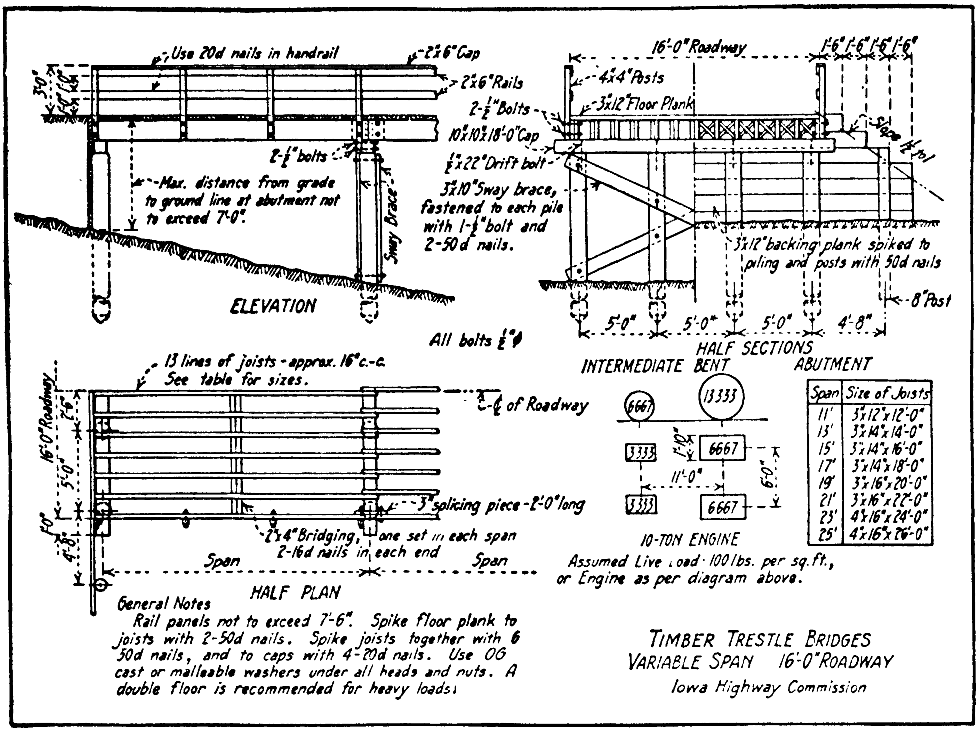


FIG. 13. TIMBER TRESTLE BRIDGE. IOWA HIGHWAY COMMISSION.

DETAIL SPECIFICATIONS.

12. **Piles.**—Piles shall be carefully selected to suit the place and ground where they are to be driven. When required by the engineer, pile butts shall be banded with iron or steel for driving, and the tips with suitable iron or steel shoes; such shoes will be furnished by the railroad company.

13.—Piles shall be driven to a firm bearing, satisfactory to the engineer, or until five blows of a hammer weighing 3,000 lb., falling 15 feet (or a hammer and fall producing the same mechanical effect), are required to cause an average penetration of one-half (½) in. per blow, except in soft bottom, where special instructions will be given.

14.—Batter piles shall be driven to the inclination shown by the plans, and shall require but slight bending before framing.

15.—Butts of all piles in a bent shall be sawed off to one plane and trimmed so as not to leave any horizontal projection outside of the cap.

16.—Piles injured in driving, or driven out of place, shall either be pulled out or cut off, and replaced by new piles.

17. **Caps.**—Caps shall be sized over the piles or posts to a uniform thickness and even bearing on piles or posts. The side with most sap shall be placed downward.

18. **Posts.**—Posts shall be sawed to proper length for their position (vertical or batter), and to an even bearing on cap and sill.

19. **Sills.**—Sills shall be sized at the bearing of posts to one plane.

20. **Sway Braces.**—Sway bracing shall be properly framed and securely fastened to piles or posts. When necessary for pile bents, filling pieces shall be used between the braces and the piles on account of the variation in size of piles, and securely fastened and faced to obtain a bearing against all piles.

21. **Longitudinal Braces.**—Longitudinal X-braces shall be properly framed and securely fastened to piles or posts.

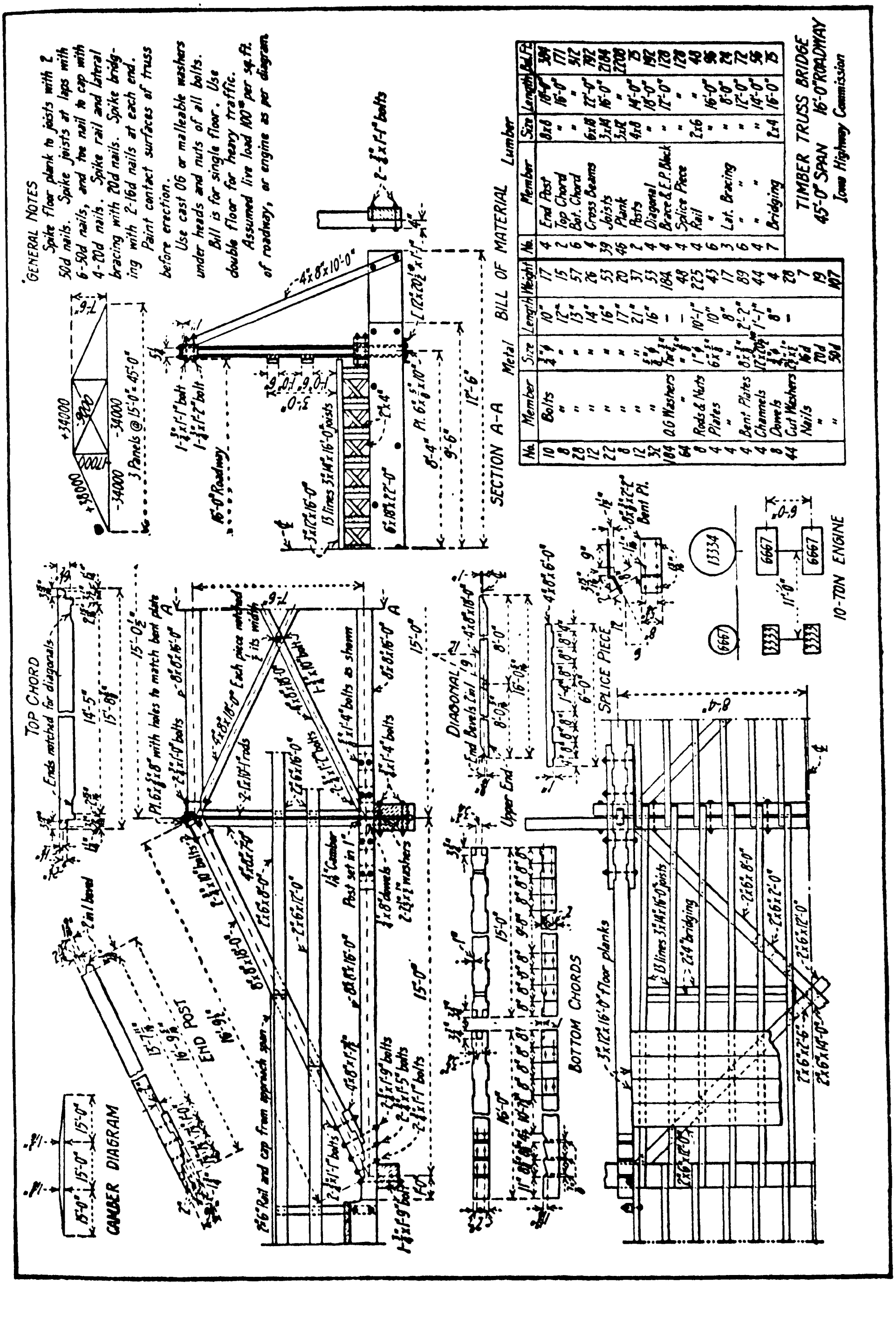


FIG. 14. TIMBER TRUSS BRIDGE. IOWA HIGHWAY COMMISSION.

22. **Girts.**—Girts shall be properly framed and securely fastened to caps, sub-sills, posts or piles, as the plans may require.

23. **Stringers.**—Stringers shall be sized to a uniform height at supports. The edges with most sap shall be placed downward.

24. **Jack Stringers.**—Jack stringers, if required on the plans, shall be neatly framed on caps, and their tops shall be in the same plane as the track stringers.

25. **Ties.**—Ties shall be framed to a uniform thickness over bearings, and shall be placed with the rough side upward. They shall be spaced regularly, cut to even length and line, as called for on the plans.

26. **Guard Rails.**—Timber guard rails shall be framed as called for on the plans, laid to line and to a uniform top surface. They shall be firmly fastened to the ties as required.

27. **Bulkheads.**—Bulkheads shall be of sufficient dimensions to keep the embankment clear of the caps, stringers and ties, at the end bents of the trestle. There shall be a space not less than two (2) in. between the back of end bent and the face of the bulkhead. The projecting ends of the bulkhead shall be sawed off to conform to the slope of the embankment, unless otherwise specified.

28. **Time of Completion.**—The work shall be completed in all its parts on or before .... A. D. 19....

29. **Payments.**—Payments will be made under the usual regulations of the railroad company.

SPECIFICATIONS FOR METAL DETAILS USED IN WOODEN BRIDGES AND TRESTLES.

30. **Wrought-iron.**—Wrought-iron shall be double-rolled, tough, fibrous and uniform in character. It shall be thoroughly welded in rolling and be free from surface defects. When tested in specimens of standard form shall give an ultimate strength of at least 50,000 lb. per sq. in., an elongation of 18 per cent in 8 in., with fracture wholly fibrous. Specimens shall bend cold, with the fiber, through 135 degrees, without sign of fracture, around a pin the diameter of which is not over twice the thickness of the piece tested. When nicked and bent, the fracture shall show at least 90 per cent fibrous.

31. **Steel.**—Steel shall be made by the open-hearth process and shall be of uniform quality It shall contain not more than 0.05 per cent sulphur; if made by the acid process it shall contain not more than 0.06 per cent phosphorus, and if made by the basic process not more than 0.04 per cent phosphorus. When tested in specimens of standard form, or full sized pieces of the same length, it shall have a desired ultimate tensile strength of 60,000 lb. per sq. in. If the ultimate strength varies more than 4,000 lb. from that desired, a retest shall be made on the same gage, which to be acceptable, shall be within 5,000 lb. of the desired ultimate. It shall have a minimum percentage of elongation in 8 in. of $\frac{1{,}500{,}000}{\text{ult. tens. strength}}$ and shall bend cold without fracture 180 degrees flat. The fracture for tensile tests shall be silky.

32. **Castings.**—Except where chilled iron is specified, castings shall be made of tough gray iron, with sulphur not over 0.10 per cent. They shall be true to pattern, out of wind and free from flaws and excessive shrinkage. If tests are demanded, they shall be made on the "Arbitration Bar" of the American Society for Testing Materials, which is a round bar 1¼ in. in diameter and 15 in. long. The transverse test shall be made on a supported length of 12 in., with load at middle. The minimum breaking load so applied shall be 2,900 lb., with a deflection of at least $\frac{1}{10}$ in. before rupture.

33. **Bolts.**—Bolts shall be of wrought-iron or steel, made with square heads, standard size, the length of thread to be 2½ times the diameter of bolt. The nuts shall be made square, standard size, with thread fitting closely the thread of bolt. Threads shall be cut according to U. S. standards.

34. **Drift Bolts.**—Drift bolts shall be of wrought-iron or steel, with or without square head, pointed or without point, as may be called for on the plans.

35. **Spikes.**—Spikes shall be of wrought-iron or steel, square or round, as called for on the plans; steel wire spikes, when used for spiking planking, shall not be used in lengths more than 6 in.; if greater lengths are required, wrought or steel spikes shall be used.

36. **Packing Spools or Separators.**—Packing spools or separators shall be of cast-iron, made to size and shape called for on plans; the diameter of the hole shall be ⅛ in. larger than diameter of packing bolts.

37. **Cast Washers.**—Cast washers shall be of cast-iron. The diameter shall be not less than 3½ times the diameter of bolt for which it is used, and its thickness equal to the diameter of bolt; the diameter of hole shall be ⅛ in. larger than the diameter of the bolt.

38. **Wrought Washers.**—Wrought washers shall be of wrought-iron or steel, the diameter shall be not less than 3½ times the diameter of bolt for which it is used, and not less than ¼ in thick. The hole shall be ⅛ in. larger than the diameter of the bolt.

39. **Special Castings.**—Special castings shall be made true to pattern, without wind, free from flaws and excessive shrinkage, size and shape to be as called for by the plans.

WORKING UNIT-STRESSES FOR STRUCTURAL TIMBER EXPRESSED IN POUNDS PER SQUARE INCH.*

Note.—The working unit-stresses given in Table V are intended for railroad bridges and trestles. For highway bridges and trestles the unit-stresses may be increased twenty-five (25) per cent. For buildings and similar structures, in which the timber is protected from the weather and practically free from impact, the unit stresses may be increased fifty (50) per cent. To compute the deflection of a beam under long-continued loading instead of that when the load is first applied, only fifty per cent of the corresponding modulus of elasticity given in the table is to be employed.

TABLE V.

UNIT STRESSES FOR STRUCTURAL TIMBER EXPRESSED IN POUNDS PER SQUARE INCH.
AMERICAN RAILWAY ENGINEERING ASSOCIATION.

Kind of Timber.	Bending.			Shearing.				Compression.						Ratio of Length of Stringer to Depth.
	Extreme Fiber Stress.		Modulus of Elasticity.	Parallel to Grain.		Longitudinal Shear in Beams.		Perpendicular to Grain.		Parallel to Grain.		For Col's under 15 Diam., Safe Stress.	Formulas for Safe Stress in Long Columns over 15 Diams.	
	Average Ultimate.	Safe Stress.	Average.	Average Ultimate.	Safe Stress.	Average Ultimate.	Safe Stress.	Elastic Limit.	Safe Stress.	Average Ultimate.	Safe Stress.			
Douglas fir.....	6100	1200	1,510,000	690	170	270	110	630	310	3600	1200	900	$1200\left(1-\frac{l}{60d}\right)$	10
Longleaf pine ...	6500	1300	1,610,000	720	180	300	120	520	260	3800	1300	980	$1300\left(1-\frac{l}{60d}\right)$	10
Shortleaf pine...	5600	1100	1,480,000	710	170	330	130	340	170	3400	1100	830	$1100\left(1-\frac{l}{60d}\right)$	10
White pine.....	4400	900	1,130,000	400	100	180	70	290	150	3000	1000	750	$1000\left(1-\frac{l}{60d}\right)$	10
Spruce.........	4800	1000	1,310,000	600	150	170	70	370	180	3200	1100	830	$1100\left(1-\frac{l}{60d}\right)$	....
Norway pine....	4200	800	1,190,000	590*	130	250	100	...	150	2600†	800	600	$800\left(1-\frac{l}{60d}\right)$	....
Tamarack......	4600	900	1,220,000	670	170	260	100	...	220	3200†	1000	750	$1000\left(1-\frac{l}{60d}\right)$	....
Western hemlock	5800	1100	1,480,000	630	160	270†	100	440	220	3500	1200	900	$1200\left(1-\frac{l}{60d}\right)$	....
Redwood.......	5000	900	800,000	300	80	....	...	400	150	3300	900	680	$900\left(1-\frac{l}{60d}\right)$	....
Bald cypress....	4800	900	1,150,000	500	120	....	...	340	170	3900	1100	830	$1100\left(1-\frac{l}{60d}\right)$	....
Red cedar......	4200	800	860,000	....	...	....	...	470	230	2800	900	680	$900\left(1-\frac{l}{60d}\right)$	....
White oak......	5700	1100	1,150,000	840	210	270	110	920	450	3500	1300	980	$1300\left(1-\frac{l}{60d}\right)$	12

Note.—These unit stresses are for a green condition of timber and are to be used without increasing the live load stresses for impact.

REFERENCES.—For additional details and information the following references may be consulted :

Foster's "A Treatise on Wooden Trestle Bridges," John Wiley & Sons, gives data and details of the design of timber trestles.

Jacoby's "Structural Details ; Design of Heavy Framing," John Wiley & Sons, gives data and details of the design of timber trestles and timber structures, and is the best book on timber construction. Every engineer interested in the design of timber structures should have a copy of Jacoby's "Structural Details."

* Adopted, Am. Ry. Eng. Assoc., Vol. 10, 1909.

† Partially air-dry. l = length in inches. d = least side in inches.

CHAPTER VIII.

STEEL BINS.

Stresses in Bin Walls.—The problem of the calculation of pressures on bin walls is similar to the problem of the calculation of pressures on retaining walls; but in the case of bin walls the material is limited in extent and the condition of static equilibrium is disturbed by drawing the material from the bottom of the bin. For plane bin walls where the plane of rupture cuts the free surface of the material (shallow bins), the formulas developed for retaining walls are directly applicable if friction on the wall is considered. The graphic solution will be found the simplest and most direct for any particular case. The following analyses of the calculations of stresses in bins have been abstracted from the author's "The Design of Walls, Bins and Grain Elevators," third edition, 1918.

STRESSES IN SHALLOW BINS.—The problem of the calculation of the pressures on bin walls is the same as the problem of the calculation of pressures on retaining walls. The forces acting on bin walls depend upon the weight, angle of repose, moisture, etc., of the material, which are variable factors, but are less variable than for the filling of retaining walls.

Algebraic Solution.—The same nomenclature will be used as in retaining walls except that P' will be used to indicate the pressure obtained by means of Cain's formulas when $z = \phi'$, N' will indicate the normal component of P', and N will indicate the normal pressure on the wall when $\phi' = 0$. This analysis applies to shallow bins, only.*

Case 1. Vertical Wall, Surface Level. Angle $z = \phi'$. Fig. 1.

$$P' = \tfrac{1}{2}w \cdot h^2 \frac{\cos^2 \phi}{\cos \phi' \left(1 + \sqrt{\dfrac{\sin(\phi + \phi') \sin \phi}{\cos \phi'}}\right)^2} \tag{1}$$

$$N' = P' \cdot \cos \phi' \tag{2}$$

If

$$\phi' = \phi$$

$$P' = \tfrac{1}{2}w \cdot h^2 \frac{\cos \phi}{(1 + \sin \phi \sqrt{2})^2} \tag{3}$$

$$N' = P' \cdot \cos \phi \tag{4}$$

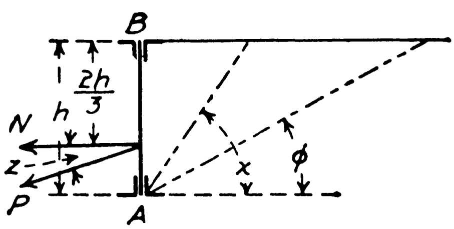


FIG. 1.

If $\phi' = 0$, which corresponds to a smooth wall,

$$N = \tfrac{1}{2}w \cdot h^2 \cdot \tan^2 (45° - \phi/2) \tag{5}$$

* A shallow bin is one where the plane of rupture cuts the free surface of the filling.

TABLE I.

CONSTANTS FOR STEEL PLATE BINS, CASE 1.

Material.	ϕ Degrees.	ϕ' Degrees.	W Lb. Per Cu. Ft.	P' Lb.	N' Lb.	N Lb.
Bituminous coal	35	18	50	$6.13h^2$	$5.83h^2$	$6.75h^2$
Anthracite coal	27	16	52	$8.73h^2$	$8.39h^2$	$9.77h^2$
Sand	34	18	90	$11.50h^2$	$10.93h^2$	$12.72h^2$
Ashes	40	31	40	$4.02h^2$	$3.44h^2$	$4.34h^2$

Case 2. Vertical Wall, Surface Surcharged at Angle δ. Angle $z = \phi'$. Fig. 2.

$$P' = \tfrac{1}{2}w\cdot h^2 \frac{\cos^2\phi}{\cos\phi'\left(1+\sqrt{\dfrac{\sin(\phi+\phi')\sin(\phi-\delta)}{\cos\phi'\cdot\cos\delta}}\right)^2} \tag{6}$$

$$N' = P'\cdot\cos\phi' \tag{7}$$

If

$$\delta = \phi$$

$$P' = \tfrac{1}{2}w\cdot h^2\frac{\cos^2\phi}{\cos\phi'} \tag{8}$$

$$N' = P'\cdot\cos\phi' = \tfrac{1}{2}w\cdot h^2\cdot\cos^2\phi \tag{9}$$

If

$$\phi' = 0$$

$$N = \tfrac{1}{2}w\cdot h^2\cdot\cos^2\phi \tag{10}$$

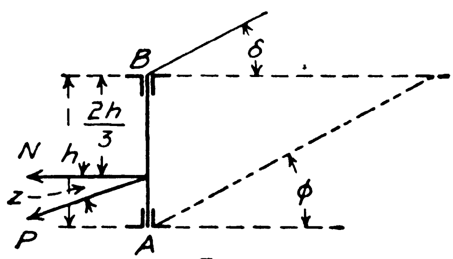


FIG. 2.

TABLE II.

CONSTANTS FOR STEEL PLATE BINS, CASE 2. $\delta = \phi$.

Material.	ϕ Degrees.	ϕ' Degrees.	W Lb. Per Cu. Ft.	P' Lb.	N' Lb.	N Lb.
Bituminous coal	35	18	50	$17.65h^2$	$16.75h^2$	$16.75h^2$
Anthracite coal	27	16	52	$21.45h^2$	$20.50h^2$	$20.50h^2$
Sand	34	18	90	$32.50h^2$	$30.90h^2$	$30.90h^2$
Ashes	40	31	40	$13.70h^2$	$11.73h^2$	$11.73h^2$

Case 3. Vertical Wall, Surcharge Negative = δ. Angle $z = \phi'$. Fig. 3.

$$P' = \tfrac{1}{2}w\cdot h^2 \frac{\cos^2\phi}{\cos\phi'\left(1+\sqrt{\dfrac{\sin(\phi+\phi')\sin(\phi+\delta)}{\cos\phi'\cdot\cos\delta}}\right)^2} \tag{11}$$

$$N' = P'\cdot\cos\phi' \tag{12}$$

If

$$\phi' = 0$$

$$N = \tfrac{1}{2} w \cdot h^2 \frac{\cos^2 \phi}{\left(1 + \sqrt{\dfrac{\sin \phi \sin (\phi + \delta)}{\cos \delta}}\right)^2} \tag{13}$$

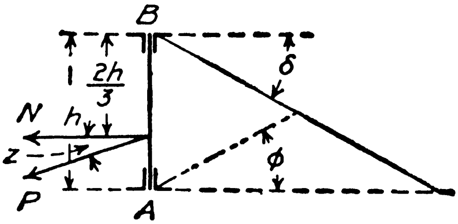


Fig. 3.

TABLE III.

Constants for Steel Plate Bins, Case 3. $\delta = -\phi$.

Material.	ϕ Degrees.	ϕ' Degrees.	W Lb. Per Cu. Ft.	P' Lb.	N' Lb.	N Lb.
Bituminous coal........	35	18	50	$4.49h^2$	$4.27h^2$	$5.13h^2$
Anthracite coal........	27	16	52	$6.64h^2$	$6.38h^2$	$7.64h^2$
Sand................	34	18	90	$8.44h^2$	$8.00h^2$	$9.61h^2$
Ashes................	40	31	40	$2.85h^2$	$2.45h^2$	$3.23h^2$

Case 4. Wall Sloping Outward. $\theta < 90° + \phi'$. *Surface Level.* Fig. 4.

$$P' = \tfrac{1}{2} w \cdot h^2 \frac{\sin^2 (\theta - \phi)}{\sin (\phi' + \theta) \sin^2 \theta \left(1 + \sqrt{\dfrac{\sin (\phi + \phi') \sin \phi}{\sin (\phi' + \theta) \sin \theta}}\right)^2} \tag{14}$$

$$N' = P' \cdot \cos \phi \tag{15}$$

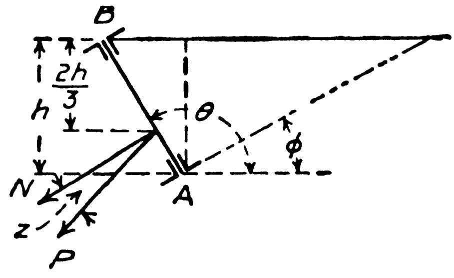


Fig. 4.

Case 5. Wall Sloping Outward. $\theta < 90° + \phi'$. *Surface Surcharged.* Fig. 5.

$$P' = \tfrac{1}{2} w \cdot h^2 \frac{\sin^2 (\theta - \phi)}{\sin (\phi' + \theta) \sin^2 \theta \left(1 + \sqrt{\dfrac{\sin (\phi + \phi') \sin (\phi - \delta)}{\sin (\phi' + \theta) \sin (\theta - \delta)}}\right)^2} \tag{16}$$

$$N' = P' \cdot \cos \phi' \tag{17}$$

Case 6. Wall Sloping Outward. $\theta > 90° + \phi'$. *Surface Level.* Fig. 6.

$$P = \tfrac{1}{2}w\cdot h^2\cdot \tan^2(45° - \phi/2)$$

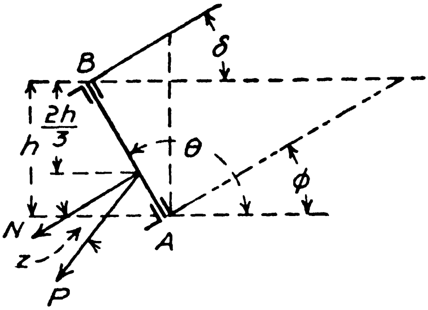


FIG. 5.

$$W = weight\ \triangle ABC = \tfrac{1}{2}w\cdot \tan\theta\cdot h^2$$

$$E = \sqrt{W^2 + P^2}$$

$$= \tfrac{1}{2}w\cdot h^2\sqrt{\tan^2\theta + \tan^4(45° - \phi/2)} \tag{18}$$

$$\tan(\theta + z - 90°) = \frac{\tan\theta}{\tan^2(45° - \phi/2)} \tag{19}$$

$$Q = E\cdot \cos z$$

$$T = E\cdot \sin z$$

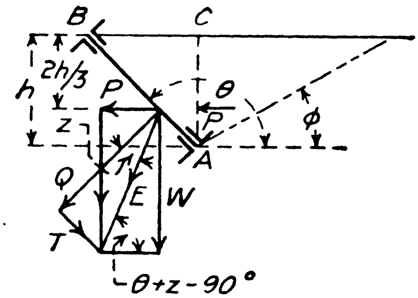


FIG. 6.

For a wall sloping outwards, and sloping surface the use of formulas is cumbersome and the calculations can be more easily made by graphic methods as explained on succeeding pages.

Tables of Pressure on Vertical Bin Walls.—The normal pressure on vertical bin walls as calculated by the preceding formulas for bituminous coal, anthracite coal, sand, and ashes are given in Table IV, Table V, Table VI, and Table VII, respectively. In the tables column 1 gives the normal pressure for a smooth vertical wall and horizontal surcharge, while column 4 gives the normal pressure on a rough wall with an angle of friction $= \phi'$. Column 2 gives the normal pressure for a smooth vertical wall and a surcharge $= \phi$, while column 5 gives the normal pressure on a rough wall with an angle of friction $= \phi'$. Column 3 gives the normal pressure for a smooth vertical wall and a negative surcharge $= -\phi$, while column 6 gives the normal pressure on a rough wall with an angle of friction $= \phi'$. It will be seen that the pressures in columns 2 and 5 are identical. For a vertical wall with $\delta = \phi$, the normal pressures as given by Rankine's and Cain's formulas are identical.

These tables have been taken from the author's "The Design of Walls, Bins and Grain Elevators." The tables of pressures and the formulas were first published in a modified form by Mr. R. W. Dull, in Engineering News.

The total pressures are given for a wall one foot long in all cases.

Note.—These tables apply to shallow bins only (bins where the plane of rupture cuts the free surface of the filling). For the calculation of the stresses in deep bins (bins where the plane of rupture cuts the side of the bin) see Chapter IX, Steel Grain Elevators.

TABLE IV.

TOTAL PRESSURE IN POUNDS FOR DEPTH "h" FOR BITUMINOUS COAL. WALL ONE FOOT LONG.

$w = 50$ lb., $\phi = 35°$.

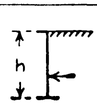
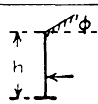
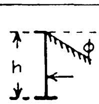
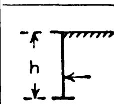
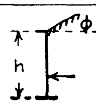
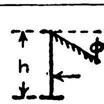
Depth, h, in Feet.	Smooth Wall, $\phi' = 0$.			Rough Wall, Angle of Friction $= \phi' = 18°$.		
	1	2	3	4	5	6
						
	$\phi' = 0$	$\delta = \phi$	$\delta = -\phi$	$\phi' = 18°$	$\delta = \phi$	$\delta = -\phi$
1	6.75	16.75	5.83	5.83	16.75	4.27
2	27	67	20.5	23.32	67	17.1
3	60.75	150.75	46.2	52.47	150.75	38.4
4	108	268	82	93.4	268	68.3
5	168.75	418.75	128	145.7	418.75	107
6	243	603	184.5	209.4	603	156
7	333	821	257	286	821	209
8	432	1,072	328	373	1,072	273
9	547	1,357	415	472	1,357	346
10	675	1,675	513	583	1,675	427
11	817	2,027	615	705	2,027	516
12	972	2,412	738	840	2,412	615
13	1,141	2,831	866	985	2,831	722
14	1,323	3,283	1,005	1,143	3,283	838
15	1,519	3,769	1,152	1,312	3,769	960
16	1,728	4,288	1,311	1,492	4,288	1,093
17	1,951	4,841	1,480	1,685	4,841	1,232
18	2,187	5,427	1,660	1,889	5,427	1,382
19	2,437	6,047	1,852	2,105	6,047	1,541
20	2,700	6,700	2,052	2,332	6,700	1,708
21	2,977	7,387	2,262	2,571	7,387	1,883
22	3,267	8,102	2,483	2,821	8,102	2,067
23	3,571	8,861	2,560	3,084	8,861	2,259
24	3,888	9,648	2,810	3,358	9,648	2,460
25	4,219	10,469	3,206	3,644	10,469	2,669
26	4,563	11,323	3,468	3,941	11,323	2,887
27	4,923	12,211	3,740	4,250	12,211	3,113
28	5,292	13,142	4,022	4,570	13,142	3,348
29	5,677	14,087	4,314	4,903	14,087	3,591
30	6,075	15,075	4,617	5,247	15,075	3,843

TABLE V.

TOTAL PRESSURE IN POUNDS FOR DEPTH "h" FOR ANTHRACITE COAL.
WALL ONE FOOT LONG.

$w = 52$ lb., $\phi = 27°$.

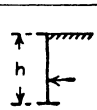
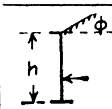
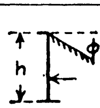
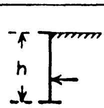
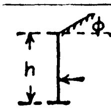
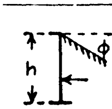
Depth, h, in Feet.	Smooth Wall, $\phi' = 0$.			Rough Wall, Angle of Friction $= \phi' = 16°$		
	1	2	3	4	5	6
						
	$\delta' = 0$	$\delta = \phi$	$\delta = -\phi$	$\phi' = 16°$	$\delta = \phi$	$\delta = -\phi$
1	9.75	20.5	7.64	8.39	20.5	6.38
2	39.0	82.0	30.6	33.5	82.0	25.5
3	87.8	184.5	68.8	75.5	184.5	57.5
4	156	328	122.2	134.2	328	102.0
5	244	513	191	210	513	159.5
6	351	738	267	302	738	230
7	478	1,005	374	411	1,005	313
8	624	1,312	489	536	1,312	402
9	790	1,661	619	680	1,661	517
10	975	2,050	764	839	2,050	638
11	1,180	2,481	925	1,014	2,481	773
12	1,405	2,952	1,100	1,209	2,952	920
13	1,648	3,465	1,290	1,418	3,465	1,080
14	1,910	4,018	1,497	1,643	4,018	1,250
15	2,193	4,613	1,720	1,887	4,613	1,436
16	2,500	5,248	1,953	2,145	5,248	1,636
17	2,808	5,945	2,207	2,421	5,945	1,845
18	3,160	6,642	2,471	2,718	6,642	2,064
19	3,521	7,400	2,758	3,030	7,400	2,310
20	3,902	8,200	3,053	3,350	8,200	2,554
21	4,303	9,041	3,372	3,700	9,041	2,820
22	4,718	9,922	3,701	4,061	9,922	3,086
23	5,156	10,845	4,040	4,438	10,845	3,372
24	5,611	11,808	4,398	4,833	11,808	3,680
25	6,097	12,813	4,770	5,244	12,813	3,985
26	6,600	13,858	5,160	5,672	13,858	4,521
27	7,112	14,945	5,560	6,116	14,945	4,650
28	7,638	16,072	5,979	6,578	16,072	5,000
29	8,202	17,241	6,421	7,056	17,241	5,370
30	8,775	18,450	6,880	7,551	18,450	5,742

TABLE VI.

Total Pressure in Pounds for Depth "h" for Sand. Wall One Foot Long.

$w = 90$ lb., $\phi = 34°$.

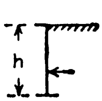
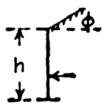
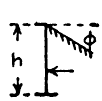
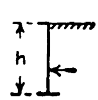
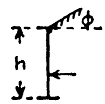
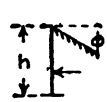
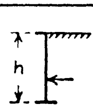
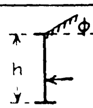
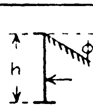
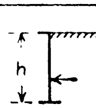
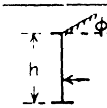
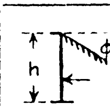
Depth, h, in Feet.	Smooth Wall, $\phi' = 0$.			Rough Wall, Angle of Friction $= \phi' = 18°$.		
	1	2	3	4	5	6
						
	$\phi' = 0$	$\delta = \phi$	$\delta = -\phi$	$\phi' = 18°$	$\delta = \phi$	$\delta = -\phi$
1	12.72	30.9	9.61	10.93	30.9	8
2	50.8	123.6	38.4	43.7	123.6	32
3	114.5	278	86.40	98.5	278	72
4	203.7	494	113.8	175	494	128
5	318	772	240	273	772	200
6	458	1,113	346	394	1,113	288
7	624	1,515	471	535	1,515	392
8	815	1,980	615	700	1,980	512
9	1,030	2,500	778	885	2,500	648
10	1,272	3,090	961	1,093	3,090	800
11	1,540	3,740	1,162	1,345	3,740	968
12	1,833	4,450	1,383	1,575	4,450	1,152
13	2,150	5,230	1,624	1,848	5,230	1,352
14	2,495	6,060	1,880	2,160	6,060	1,568
15	2,862	6,960	2,160	2,460	6,960	1,800
16	3,256	7,910	2,460	2,798	7,910	2,048
17	3,676	8,930	2,777	3,159	8,930	2,312
18	4,121	10,012	3,114	3,541	10,012	2,592
19	4,592	11,155	3,469	3,946	11,155	2,888
20	5,088	12,360	3,844	4,372	12,360	3,200
21	5,610	13,627	4,238	4,820	13,627	3,528
22	6,156	14,956	4,651	5,290	14,956	3,872
23	6,729	16,346	5,084	5,782	16,346	4,232
24	7,327	17,798	5,535	6,296	17,798	4,608
25	7,950	19,313	6,006	6,831	19,313	5,000
26	8,599	20,889	6,496	7,389	20,889	5,408
27	9,273	22,526	7,006	7,968	22,526	5,832
28	9,972	24,225	7,534	8,569	24,225	6,272
29	10,698	25,987	8,082	9,192	25,987	6,728
30	11,448	27,810	8,649	9,837	27,810	7,200

TABLE VII.

Total Pressure in Pounds for Depth "h" for Ashes.
Wall One Foot Long.

$w = 40$ lb., $\phi = 40°$.

Depth, h, in Feet.	Smooth Wall, $\phi' = 0$.			Rough Wall, Angle of Friction $= \phi' = 31°$.		
	1	2	3	4	5	6
						
	$\phi' = 0$	$\delta = \phi$	$\delta = -\phi$	$\phi' = 31°$	$\delta = \phi$	$\delta = -\phi$
1	4.35	11.73	3.23	3.44	11.73	2.45
2	17.4	47	12.9	13.76	47	9.80
3	39.2	105.7	29.01	30.96	105.7.	22.05
4	69.6	188	31.7	55.04	188	39.20
5	108.7	294	80.8	86	294	61.2
6	156.4	423	116	124	423	88.2
7	213	576	158	168	576	120
8	278	751	207	220	751	157
9	352	952	261	279	952	199
10	435	1,173	323	344	1,173	245
11	526	1,420	391	416	1,420	296
12	626	1,690	465	495	1,690	353
13	735	1,985	546	581	1,985	414
14	852	2,300	634	674	2,300	480
15	978	2,640	726	774	2,640	550
16	1,113	3,010	828	881	3,010	627
17	1,257	3,400	934	994	3,400	708
18	1,408	3,803	1,045	1,115	3,803	794
19	1,527	4,240	1,165	1,242	4,240	884
20	1,740	4,700	1,290	1,376	4,700	980
21	1,920	5,181	1,423	1,517	5,181	1,080
22	2,100	5,677	1,561	1,665	5,677	1,186
23	2,300	6,215	1,706	1,820	6,215	1,296
24	2,506	6,756	1,860	1,981	6,756	1,411
25	2,720	7,331	2,017	2,150	7,331	1,531
26	2,940	7,929	2,180	2,325	7,929	1,656
27	3,165	8,551	2,352	2,508	8,551	1,786
28	3,406	9,196	2,530	2,697	9,196	1,921
29	3,660	9,865	2,718	2,893	9,865	2,060
30	3,915	10,557	2,910	3,096	10,557	2,205

STRESSES IN SHALLOW BINS, Graphic Solution.—The graphic solution will be given for two cases which frequently occur in practice.

Graphic Solution. Hopper Bin, Level Full.*—The calculation of stresses in bins by means of graphics will be illustrated by the following problem taken from "The Design of Walls, Bins and Grain Elevators." A cross-section of the bin shown in Fig. 7 is filled with coal weighing 58 lb. per cu. ft., and having an angle of repose $\phi = 30°$. The total pressure on the plane $A-H$ is

$$P_1 = \tfrac{1}{2}w \cdot h^2 \frac{1 - \sin\phi}{1 + \sin\phi} = 3{,}130 \text{ lb.}$$

acting horizontally through a point 12 ft. below the top surface. Now, to find the pressure P_2 on the plane $G-A$, produce P_1 until it intersects the line O_2 = the weight of triangle AHG = 10,440

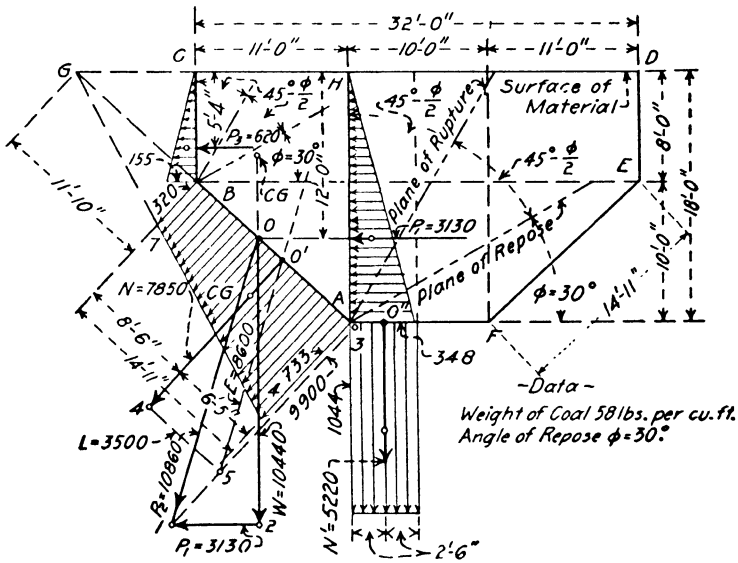


FIG. 7.

lb. at O, and by constructing $O-1 = P_2 = 10{,}860$ lb. P_2 is parallel to E in Fig. 7. The normal pressure on $A-g$ is 9,900 lb. Now $A-1 = 9{,}900$ lb. acts through the center of gravity of triangle $AG4$, and is equal to the area of $AG4 \times w$. The normal unit pressure at A is 733 lb. per sq. ft., and the normal unit pressure at B is 320 lb. per sq. ft. The normal pressure on AB acts through the center of gravity of the shaded area, and is $N = 7{,}850$ lb. Also by construction $E = 8{,}600$ lb. The pressure on bottom $A-F$ is equal to $18 \times 58 = 1{,}044$ lb. per sq. ft. The pressure on the wall $C-B$ is

$$P_3 = \tfrac{1}{2}w \cdot h^2 \frac{1 - \sin\phi}{1 + \sin\phi} = 620 \text{ lb.}$$

Calculation of Stresses in Framework.—The loads on the bin walls are carried by a transverse framework as shown in Fig. 8, spaced 17 ft. 0 in. center to center. The loads at the joints act parallel to the pressures as previously calculated, and the loads can be calculated in the same manner as for a simple beam loaded with a similar loading. The stresses are calculated by graphic resolution and by algebraic moments as shown in Fig. 8 and Fig. 9.

Hopper Bin, Top Surface Heaped.—The bin in Fig. 10 is heaped at the angle of repose, $\phi = 30°$. To calculate the pressure on side $A-B$, proceed as follows: Locate points G and H,

* The calculations are made for a section of the bin one foot long.

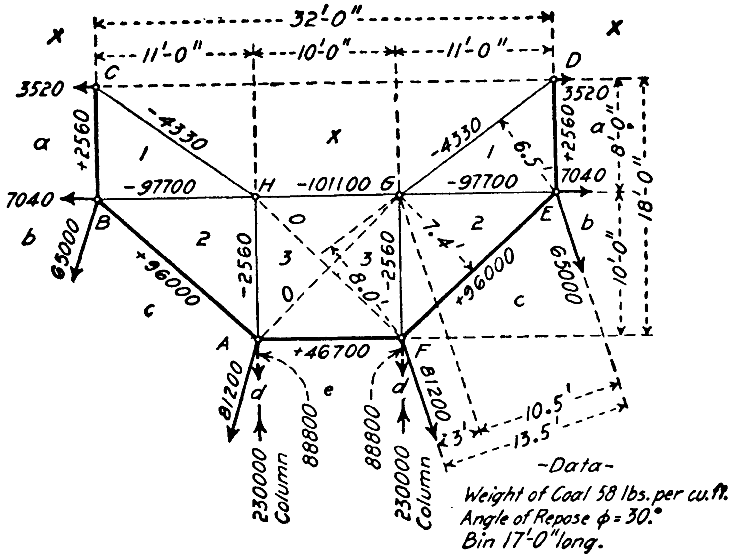


FIG. 8.

Algebraic Moments.

Center of Moments, E.

Stress GD.

$-GD \times 6.5' - 3520 \times 8' = 0$

$GD = -4330$

Stress FG.

$-FG \times 11' - 3520 \times 8' = 0$

$FG = -2560$

Center of Moments, F.

Stress GH.

$-GH \times 10' - 3520 \times 18' - 7040 \times 10' - 65000 \times 13.5' = 0$

$GH = -101100$

Stress GE.

$-GE \times 10' + 4330 \times 8' - 3520 \times 18' - 7040 \times 10' - 65000 \times 13.5 = 0 \quad GE = -97700$

Center of Moments, G.

Stress ED.

$-ED \times 11' + 3520 \times 8' = 0$

$ED = +2560$

Stress FE.

$FE \times 7.4' - 3520 \times 8' - 65000 \times 10.5 = 0$

$FE = +96000$

Stress AF.

$AF \times 10' + 81200 \times 3' - 65000 \times 10.5' - 3520 \times 8' = 0 \quad AF = +46700$

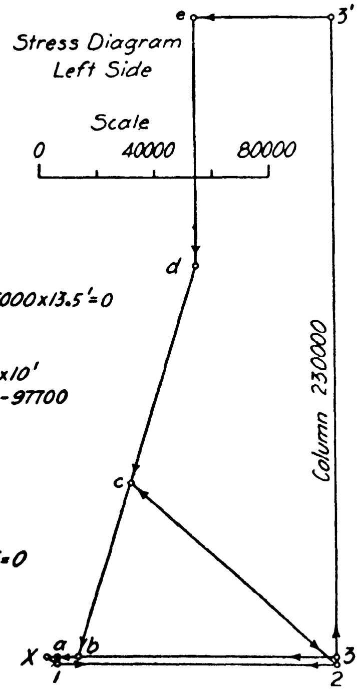


FIG. 9.

and calculate the horizontal pressure $P_1 = 7{,}680$ lb., acting on the plane $H-K$ at $\frac{1}{3}HK$ above H. Pressure P_1 was calculated by the graphic method. Produce P_1 until it intersects at O the line of action of the weight of the triangle GHK acting through the center of gravity of the triangle. From O lay off $O-1 = W = 19{,}900$ lb., acting downwards, and from 1 lay off $1-2 = P_1 = 7{,}680$ lb., acting to the left. Then $O-2 = P_2 = 21{,}300$ lb. Now P_2 = area triangle $6'GH\cdot w$, and

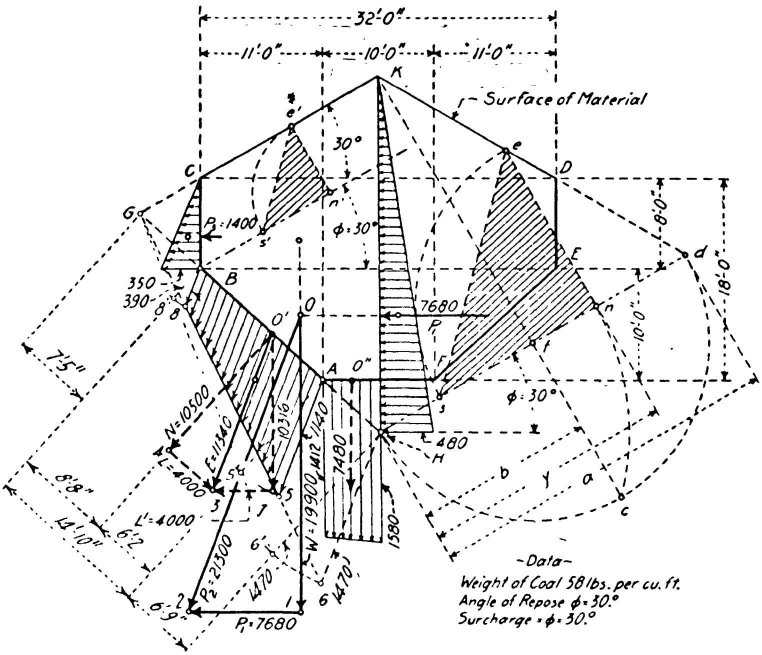


FIG. 10.

E = area $8'-B-A-5'\cdot w = 11{,}340$ lb. Force E acts through the center of gravity of area $8-B-A-5$. The horizontal pressure on plane $C-B = 1{,}400$ lb. = area $s'e'n'\cdot w$. The vertical pressure on the left-hand side of the bottom $A-F$ is 7,480 lb., acting through the center of gravity of the pressure polygon. The vertical unit pressure at A is 1,412 lb. per sq. ft.

STRESSES IN SUSPENSION BUNKERS.—The suspension bunker shown in (*a*) Fig. 11, carries a load which varies from zero at the support to a maximum at the center. If the bunker is level full the loading from the supports to the center varies nearly as the ordinates to a straight line, while if the bunker is surcharged the straight line assumption for loading is more nearly correct.

We will, therefore, assume that the loading of the bunker in (*a*) is represented by the triangular loading varying from p = zero at each support to a maximum of $p = P$ at the center.

Let l = one-half the span in feet;

S = the sag in feet;

H = the horizontal component of the stress in the plate in lb. per lineal foot of bin;

w = weight of bin filling in lb. per cu. ft.;

T = maximum tension in plate in lb. per lineal foot of bin;
V = reaction of the bunker in lb. per lineal foot of bin;
C = capacity of bunker in cu. ft. per lineal foot of bin;
B = origin of coordinates.

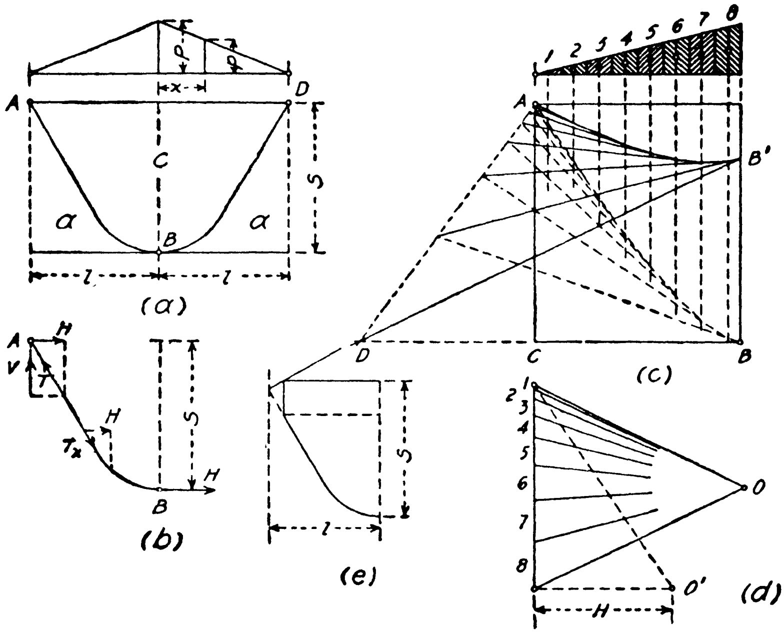


FIG. 11.

Now if the right-hand half of the bunker be cut away as in (*b*) and moments be taken about *A*, the moment will be

$$M = H \cdot S \tag{20}$$

If the bunker be assumed as an equilibrium polygon drawn by using a force polygon, the bending moment at the center is equal to the pole distance multiplied by the intercept S. Therefore H must be equal to the pole distance of the force polygon.

The following equations are deduced in the author's "The Design of Walls, Bins and Grain Elevators."

Equation of the curve of the bunker

$$y = \tfrac{1}{3}\frac{S}{l^2}\left(3x^2 - \frac{x^3}{l}\right) \tag{21}$$

Capacity of bunker level full

$$C = \tfrac{2}{3}l \cdot S \tag{22}$$

In calculating P for any given bunker, since P is the maximum pressure for a triangular loading

$$P = \frac{c \cdot w}{l} \tag{23}$$

for a bunker level full

$$P = \tfrac{2}{3}S \cdot w \tag{24}$$

also

$$H = \frac{C \cdot w \cdot l}{3S} \tag{25}$$

$$V = \tfrac{1}{2} C \cdot w$$
$$= \tfrac{2}{3} S \cdot l \cdot w, \text{ for a bin level full} \tag{26}$$

$$T = C \cdot w \sqrt{\frac{1}{4} + \frac{l^2}{9S^2}} \tag{27}$$

The length of the curve of a suspension bunker is given in Table VIII.

TABLE VIII.

LENGTH OF ONE-HALF CURVE, L.

Sag ratio = S/l.	Length, L.	Sag ratio = S/l.	Length, L.
$\frac{1}{3}$	$1.06378l$	1	$1.45722l$
$\frac{1}{2}$	$1.13686l$	$\frac{6}{5}$	$1.61131l$
$\frac{2}{3}$	$1.22992l$	$\frac{4}{3}$	$1.71906l$
$\frac{3}{4}$	$1.28307l$	$\frac{3}{2}$	$1.85815l$
$\frac{7}{8}$	$1.36651l$		

The curve may be constructed graphically as follows: In (*c*) Fig. 11 it is required to pass the curve through the points A and B. The loads 1, 2, 3, 4, etc., are laid off in the force polygon (*d*), and a pole O is taken. The equilibrium polygon $A-B'$ is then constructed in (*c*). Now we know from graphic statics that if two poles be taken for the force polygon in (*d*), and corresponding equilibrium polygons be drawn through A, the strings meeting on the same load will intersect on a line through A parallel to the line $O-O'$. Now D is determined by the intersection of rays $D-B'$ and $D-B$. The true curve is then easily constructed and pole O' is located.

If the bunker is surcharged by vertical walls as shown in (*e*) the curve is extended until it meets the slope of the material, and the span and sag are to be used as shown.

Deep Bins.—For the calculation of the stresses in deep bins, see the calculation of the stresses in grain bins, Chapter IX.

For methods of calculating the stresses in hopper bins with the top surface surcharged, and the calculation of the stresses in bin bottoms and circular girders, see the author's "The Design of Walls, Bins and Grain Elevators."

Angle of Repose.—The angle of repose and the weights of different materials are given in Table IX.

DATA.—For angles of internal friction, see Table IX, and for angles of friction on bin walls, see Table X.

TABLE IX.

WEIGHT AND ANGLE OF REPOSE OF COAL, COKE, ASHES AND ORE.

Material.	Weight Lb. per Cu. Ft.	Angle of Repose ϕ in Degrees.	Authority.
Bituminous coal..........	50	35	Link Belt Machinery Co.
Bituminous coal..........	47	35	Link Belt Engineering Co.
Bituminous coal..........	47 to 56	..	Cambria Steel.
Anthracite coal..........	52	27	Link Belt Machinery Co.
Anthracite coal..........	52.1	27	Link Belt Engineering Co.
Anthracite coal fine........	..	27	K. A. Muellenhoff.
Anthracite coal..........	52 to 56	..	Cambria Steel.
Slaked coal..............	..	45	Wellman-Seaver-Morgan Co.
Slaked coal..............	53	$37\frac{1}{2}$	Gilbert and Barth.
Coke....................	23 to 32	..	Cambria Steel.
Ashes....................	40	40	Link Belt Machinery Co.
Ashes, soft coal..........	40 to 45	..	Cambria Steel.
Ore, soft iron............	..	35	Wellman-Seaver-Morgan Co.

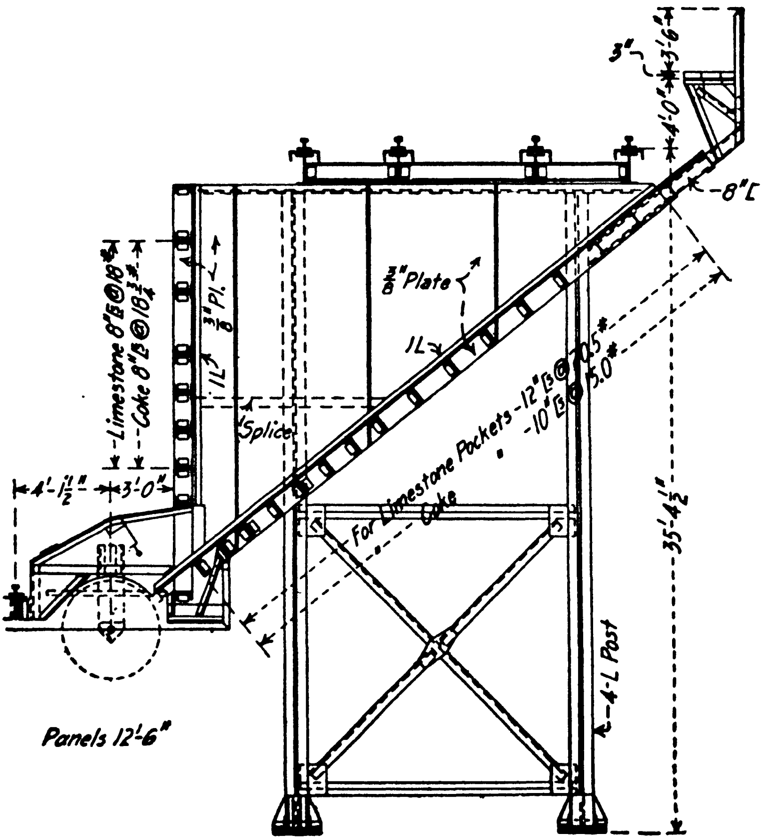
Coal, ore, etc., will give an angle of $\phi = 40°$ if the material is dry, but if the material is wet the angle of repose may be increased to nearly 90°.

Angle of Friction on Bin Walls.—The values in Table X may be used in the absence of more accurate data.

TABLE X.

ANGLE OF FRICTION OF DIFFERENT MATERIALS ON BIN WALLS.

Material.	Steel Plate. ϕ' in Degrees.	Wood Cribbed. ϕ' in Degrees.	Concrete. ϕ' in Degrees.
Bituminous coal	18	35	35
Anthracite coal	16	25	27
Ashes	31	40	40
Coke	25	40	40
Sand	18	30	30



Typical Section through Pockets.

FIG. 12. COKE AND STONE BINS, LACKAWANNA STEEL CO.

Self-cleaning Hoppers.—In order to have hoppers self-cleaning when the material is moist it is necessary to have the hopper bottoms slope at an angle considerably in excess of the angle of repose ϕ or angle of friction ϕ'.

Ore pockets on the Great Lakes are made with hopper bottoms at an angle of 48° 40′ to 50° 45′, but the majority are at an angle of 49° 45′. Bituminous coal will slide down a steel chute at an angle of 40° and a wooden chute at an angle of 45°. Anthracite coal will slide down a steel chute at an angle of 30° and down a wooden chute at an angle of 35°.

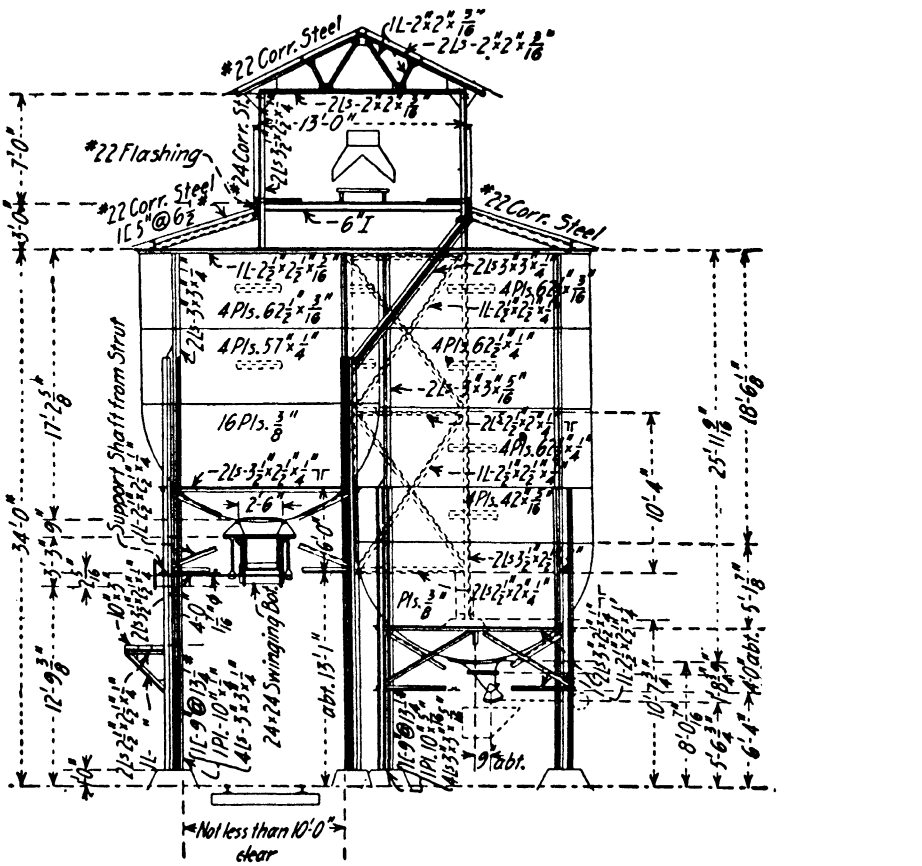


FIG. 13. ELEVATION CIRCULAR STEEL ORE BIN FOR OLD DOMINION COPPER MINING CO.

DESIGN OF BINS.—Bins are usually subjected to sudden loads and vibrations and should be designed for two-thirds the allowable unit stresses for dead loads given in §§ 33 to 41, inclusive, in "Specifications for Steel Frame Buildings," Chapter I.

Bins are made of timber, of structural steel, or of concrete, or the different materials may be used in combination.

FLAT PLATES.—The analysis of the stresses in flat plates supported or fixed at their edges is extremely difficult. The following formulas by Grashof may be used: The coefficient of lateral contraction is taken as $\frac{1}{4}$. For a full discussion of these formulas based on Grashof's "Theorie Der Elasticitat und Festigkeit" see Lanza's Applied Mechanics.

1. *Circular plate of radius r and thickness t, supported around its perimeter and loaded with w*

per square inch.—Let f = maximum fiber stress, v = maximum deflection, and E = modulus of elasticity,

$$f = \frac{117}{128} \frac{w \cdot r^2}{t^2} \tag{28}$$

$$v = \frac{189}{256} \frac{w \cdot r^4}{E \cdot t^3} \tag{29}$$

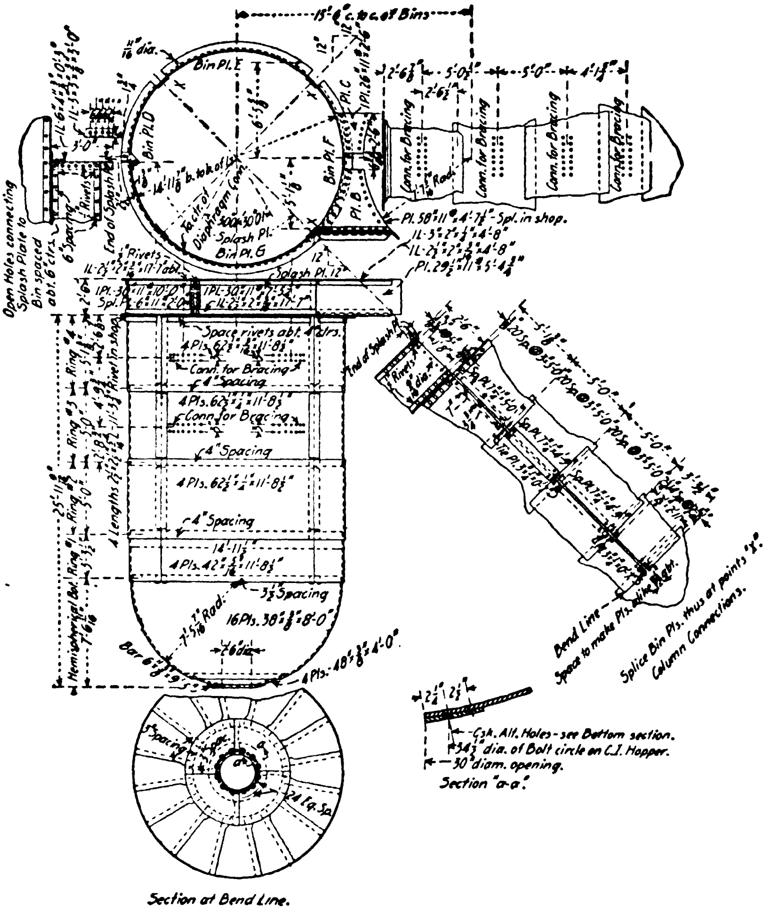


FIG. 14. DETAILS FOR CIRCULAR BINS FOR OLD DOMINION COPPER MINING CO

2. *Circular plate built in or fixed at the perimeter.*

$$f = \frac{45}{64} \frac{w \cdot r^2}{t^2} \tag{30}$$

$$v = \frac{45}{256} \frac{w \cdot r^4}{E \cdot t^3} \tag{31}$$

3. ***Rectangular plate of length a breadth b, and thickness t, built in or fixed at the edges and***

carrying a uniform load w per square inch.—Let f_a be the unit stress parallel to a, f_b be the unit stress parallel to b, and $a > b$.

$$f_a = \frac{b^4 \cdot w \cdot a^2}{2(a^4 + b^4)t^2}; \quad f_b = \frac{a^4 \cdot w \cdot b^2}{2(a^4 + b^4)t^2} \tag{32}$$

$$v = \frac{a^4 \cdot b^4 \cdot w}{(a^4 + b^4)32 E \cdot t^3} \tag{33}$$

For a square plate $a = b$,

$$f = \frac{w \cdot a^2}{4t^2} \tag{34}$$

$$v = \frac{w \cdot a^4}{64 E \cdot t^3} \tag{35}$$

The strength of plates simply supported on the edges is about ⅔ the strength of plates fixed. Plates riveted or bolted around the edges may be considered as fixed.

For a diagram giving the safe loads on flat plates, see the author's "The Design of Walls, Bins and Grain Elevators," also see Part II.

Buckle Plates.—Buckle plates are made by "dishing" flat plates as in Table 59, Part II. The width of the buckle W, or length L, varies from 2 ft. 6 in. to 5 ft. 6 in. The buckles may be turned with the greater dimension in either direction of the plate. Several buckles may be put

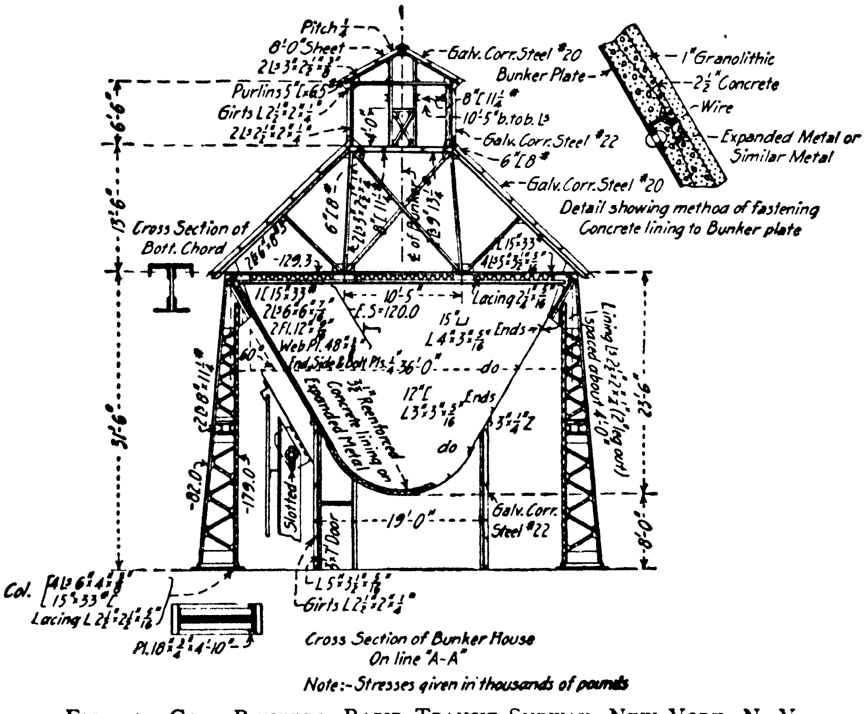


FIG. 15. COAL BUNKERS, RAPID TRANSIT SUBWAY, NEW YORK, N. Y.

in one plate, all of which must be the same size and symmetrically placed. Buckle plates are made ¼ in., $\frac{5}{16}$ in., ⅜ in. and $\frac{7}{16}$ in. in thickness. Buckle plates should be firmly bolted or riveted around the edges with a maximum spacing of 6 in., and should be supported transversely between the buckles. The process of buckling distorts the plate and an extra width should be ordered and the plate should be trimmed after the process is complete.

Strength of Buckle Plates.—The safe load for a buckle plate with buckles placed up, is approximately given by the formula

$$W = 4f \cdot R \cdot t \tag{36}$$

where W = total safe uniform load in lb.;
f = safe unit stress in lb. per sq. in.;
R = depth of buckle in in.;
t = thickness of plate in in.

Where buckle plates are riveted and the buckle placed down the safe load is from 3 to 4 times that given above.

TYPES OF BINS.—The most common types are (1) the suspension bunker, (2) the hopper bin, and (3) the circular bin.

Suspension Bunkers.—Suspension bunkers are made by suspending a steel framework from two side members, the weight of the filling causing the sides to assume the curve of an equilibrium polygon. The stresses in the plates of a true suspension bunker are pure tensile stresses. Steel suspension bunkers are commonly lined with a concrete lining about 1½ to 3½ in. thick, reinforced with wire fabric, to protect the metal of the bin.

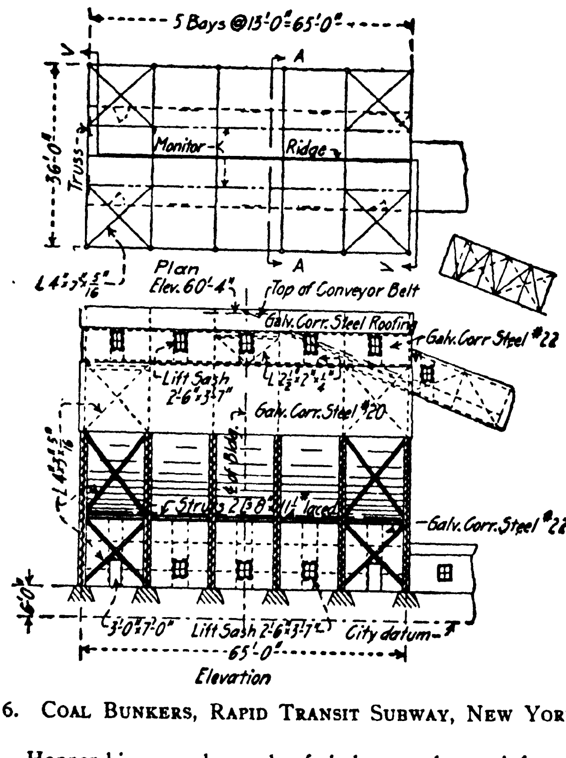


FIG. 16. COAL BUNKERS, RAPID TRANSIT SUBWAY, NEW YORK, N. Y.

Hopper Bins.—Hopper bins may be made of timber, steel, or reinforced concrete. A steel coke and stone bin, erected by the Lackawanna Steel Company, is shown in Fig. 12. These bins were divided into panels 12 ft. 6 in. center to center, with double partitions at each panel point, leaving a clear length of 11 ft. 6 in. The bins are lined throughout with ⅜ in. plates. All rivets in the floor are countersunk. The gates at the bottom of the bin are cylindrical and are revolved

by a system of shafting and gears. There is an opening in the side of the drum, and when the drum is revolved this opening comes opposite the opening in the bottom of the bin and the drum is filled. The drum is then revolved and the material is dumped into the larries.

Circular Bins.—Circular bins are made of both steel and reinforced concrete. A circular ore bin with a hemispherical bottom is shown in Fig. 13 and Fig. 14.

EXAMPLES OF BINS. Steel Coal Bin for Rapid Transit Subway.—A cross-section of a 1,000-ton suspension bunker built by the Rapid Transit Subway, New York City, is shown in Fig. 15 and Fig. 16. The bunker is supported on posts and is covered by corrugated steel. The bin is lined with a layer of concrete 3½ in. thick, reinforced with expanded metal. The details of construction are plainly shown in the cuts.

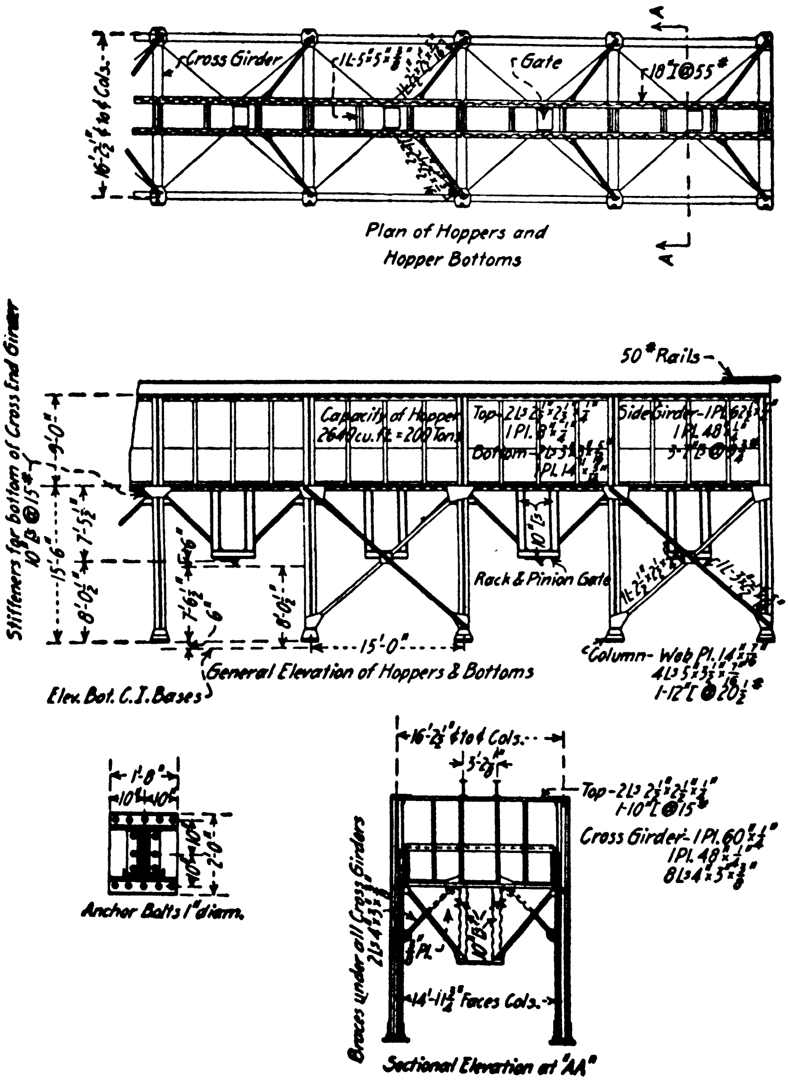


FIG. 17. HOPPER BIN CANANEA CONSOLIDATED COPPER CO., CANANEA, MEXICO.

Ore Bins for Cananea Consolidated Copper Company.—Detail drawings of a hopper ore bin built by the Cananea Consolidated Copper Company are shown in Fig. 17. The ore is coarse and heavy and is dumped from cars on the top of the bins. The ore is drawn off through gates on the bottom and is carried away on a conveyor. The side plates are $\frac{1}{2}$ in. thick and are stiffened with channels spaced about 4 ft. apart. The hopper plates are $\frac{3}{8}$ in. thick and are stiffened with 10 in. channels.

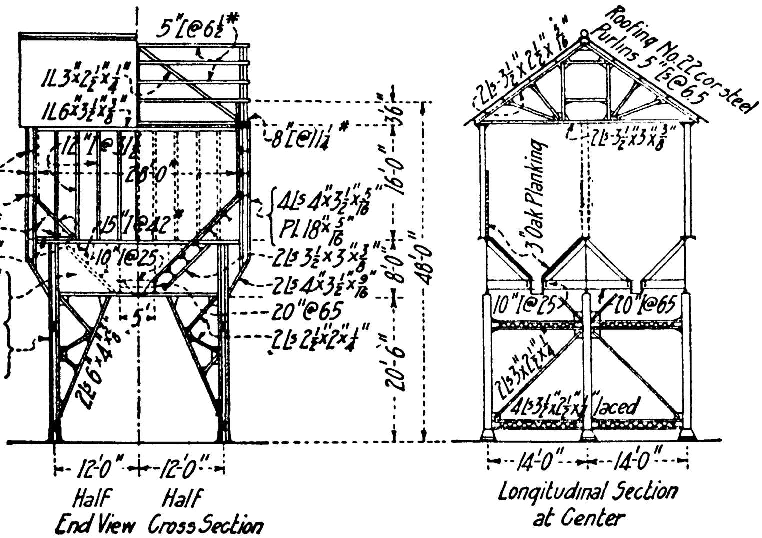


FIG. 18. STEEL COAL BINS AT COKETON, W. VA.

Steel Coal Bins for Davis Coal and Coke Co.—The steel coal bin shown in Fig. 18 was designed by the American Bridge Company for the Davis Coal and Coke Co. for the coke ovens at Coketon, W. Va. The framework is made of structural steel and is covered with corrugated steel. The bin is lined with 3 in. oak plank spiked to timber spiking pieces which are bolted to the steel beams. The bin is carried on plate girders each having a web plate 96 in. × $\frac{3}{8}$ in., and top and bottom flanges of two angles 6" × 6" × $\frac{9}{16}$". The bin is filled by a belt conveyor passing over the top of the bin, as shown in Fig. 18. The coal is drawn from the bins through gates into cars and is hauled to the coke ovens. The capacity of the bin is 300 tons.

References.—For the design of reinforced concrete bins, and for additional data and examples, see the author's "The Design of Walls, Bins and Grain Elevators."

CHAPTER IX.

STEEL GRAIN ELEVATORS.

Introduction.—Grain elevators, or "silos," as they are called in Europe, may be divided into two classes according to the arrangement of the bins and elevating machinery: (*a*) elevators which are self contained, with all the storage bins in the main elevator or working house; and (*b*) elevators having a working house containing the elevating machinery, while the storage is in bins connected with the working house by conveyors. The working house is usually rectangular in shape, with square or circular bins; while the independent storage bins are usually circular.

With reference to the materials of which they are constructed, elevators may be divided into (1) timber; (2) steel; (3) concrete; (4) tile, and (5) brick. Steel grain elevators, only, will be considered in this chapter. For a complete treatise on the design of grain elevators, see the author's "The Design of Walls, Bins and Grain Elevators."

STRESSES IN GRAIN BINS.—The problem of calculating the pressure of grain on bin walls is somewhat similar to the problem of the retaining wall, but is not so simple. The theory of Rankine will apply in the case of shallow bins with smooth walls where the plane of rupture cuts the grain surface, but will not apply to deep bins or bins with rough walls. (It should be remembered that Rankine assumes a granular mass of unlimited extent.)

Stresses in Deep Bins.—Where the plane of rupture cuts the sides of the bin the solution for shallow bins does not apply.

Nomenclature.—The following nomenclature will be used:

ϕ = angle of repose of the filling;
ϕ' = the angle of friction of the filling on the bin walls;
$\mu = \tan\phi$ = coefficient of friction of filling on filling;
$\mu' = \tan\phi'$ = coefficient of friction of filling on the bin walls;
x = angle of rupture;
w = weight of filling in lb. per cu. ft.;
V = vertical pressure of the filling in lb. per sq. ft.;
L = lateral pressure of the filling in lb. per sq. ft.;
A = area of bin in sq. ft.;
U = circumference of bin in ft.;
$R = A/U$ = hydraulic radius of bin.

Janssen's Solution.—The bin in (*a*) Fig. 1, has a uniform area A, a constant circumference U, and is filled with a granular material weighing w per unit of volume, and having an angle of repose ϕ. Let V be the vertical pressure, and L be the lateral pressure at any point, both V and L being assumed as constant for all points on the horizontal plane. (More correctly V and L will be constant on the surface of a dome as in (*b*).)

The weight of the granular material between the sections of y and $y + dy = A\cdot w\cdot dy$; the total frictional force acting upwards at the circumference will be $= L\cdot U\cdot\tan\phi'\cdot dy$; the total perpendicular pressure on the upper surface will be $= V\cdot A$; and the total pressure on the lower surface will be $= (V + dV)A$.

Now these vertical pressures are in equilibrium, and

$$V\cdot A - (V + dV)A + A\cdot w\cdot dy - L\cdot U\cdot\tan\phi'\cdot dy = 0$$

and

$$dV = \left(w - L\cdot\tan\phi'\frac{U}{A}\right)dy \qquad (1)$$

Now in a granular mass, the lateral pressure at any point is equal to the vertical pressure times k, a constant for the particular granular material, and

$$L = k \cdot V$$

Also let $A/U = R$ (the hydraulic radius), and $\tan \phi' = \mu'$.
Substituting the above in (1) we have

$$dV = \left(w - \frac{k \cdot V}{R}\mu'\right) dy$$

Now let

$$\frac{k \cdot \mu'}{R} = n \tag{2}$$

and

$$\frac{dV}{w - n \cdot V} = dy \tag{3}$$

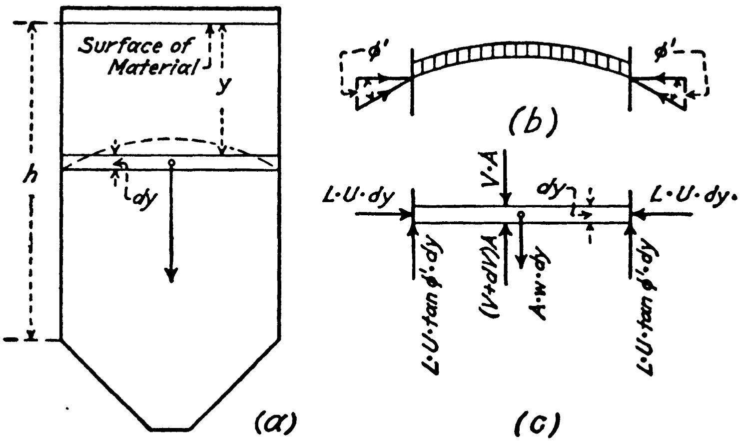


FIG. 1.

Integrating (3) we have

$$\log (w - n \cdot V) = -n \cdot y + C \tag{4}$$

Now if $y = 0$, then $V = 0$, and $C = \log w$, and (4) reduces to

$$\log \left(\frac{w - n \cdot V}{w}\right) = -n \cdot y$$

and

$$\frac{w - n \cdot V}{w} = \frac{1}{e^{n \cdot y}} = e^{-n \cdot y}$$

where e is the base of the Naperian system of logarithms. Solving for V we have

$$V = \frac{w}{n}(1 - e^{-n \cdot y}) \tag{5}$$

Substituting the value of n from (2), we have

$$V = \frac{w \cdot R}{k \cdot \mu'}(1 - e^{-k \cdot \mu' \cdot y/R}) \tag{6}$$

Now if h be taken as the depth of the granular material at any point we will have

$$V = \frac{w \cdot R}{k \cdot \mu'}(1 - e^{-k \cdot \mu' \cdot h/R}) \tag{7}$$

Also since

$$L = k \cdot V$$

$$L = \frac{w \cdot R}{\mu'}\left(1 - e^{-k \cdot \mu' \cdot h/R}\right) \qquad (8)$$

Now if w is taken in lb. per cu. ft., and R in ft., the pressure will be given in lb. per sq. ft.

For deep bins with a depth of more than two and one-half diameters the last term of the right hand member of (8) may be omitted, and

$$L' = \frac{w \cdot R}{\mu'} \text{ (approx.)} \qquad (9)$$

Now both μ' and k can only be determined by experiment on the particular grain and kind of bin. For wheat and a wooden bin, Janssen found $\mu' = 0.3$ and $k = 0.67$, making $k \cdot \mu' = 0.20$.

Jamieson found by experiment that for wheat $k = 0.6$, and he found values in Table I for μ' with wheat weighing 50 lb. per cu. ft. and having $\phi = 28°$, $\mu = 0.532$:

TABLE I.

COEFFICIENTS OF FRICTION μ' FOR WHEAT ON BIN WALLS.

JAMIESON.

Wheat Weighing 50 lb. per cu. ft., and Angle of Repose $\phi = 28$ Degrees.

Materials.	Coefficient of Friction.
Wheat on wheat	0.532
Wheat on steel trough plate bin	0.468
Wheat on steel flat plate, riveted and tie bars	0.375 to 0.400
Wheat on steel cylinders, riveted	0.365 to 0.375
Wheat on cement-concrete, smooth to rough	0.400 to 0.425
Wheat on tile or brick, smooth to rough	0.400 to 0.425
Wheat on cribbed wooden bin	0.420 to 0.450

Pleisner obtained the values of μ' as given in Table II, and of k as given in Table III.

TABLE II.

COEFFICIENTS OF FRICTION OF GRAIN BIN WALLS. PLEISNER.

Bins.	Coefficient of Friction $\mu' = \tan \phi'$.	
	Wheat.	Rye.
Cribbed bin	0.43	0.54
Ringed cribbed bin	0.58	0.78
Small plank bin	0.25	0.37
Large plank bin	0.45	0.55
Reinforced concrete bin	0.71	0.85

TABLE III.

VALUES OF $k = L/V$ FOR WHEAT AND OTHER GRAINS IN DIFFERENT BINS. PLEISNER.

Bins.	$k = L/V$.			
	Wheat.	Rye.	Rape.	Flax-seed.
Cribbed bin	0.4 to 0.5	0.23 to 0.32	....	....
Ringed cribbed bin	0.4 to 0.5	0.3 to 0.34	....	....
Small plank bin	0.34 to 0.46	0.3 to 0.45	0.5 to 0.6	0.5 to 0.6
Large plank bin	0.3	0.23 to 0.28	....	....
Reinforced concrete bin	0.3 to 0.35	0.3	....	....

TABLE IV.

HYPERBOLIC OR NAPERIAN LOGARITHMS.

N.	Log.	N.	Log.	N.	Log.
1.00	0.0000	3.65	1.2947	6.60	1.8871
1.05	0.0488	3.70	1.3083	6.70	1.9021
1.10	0.0953	3.75	1.3218	6.80	1.9169
1.15	0.1398	3.80	1.3350	6.90	1.9315
1.20	0.1823	3.85	1.3481	7.00	1.9459
1.25	0.2231	3.90	1.3610	7.20	1.9741
1.30	0.2624	3.95	1.3737	7.40	2.0015
1.35	0.3001	4.00	1.3863	7.60	2.0281
1.40	0.3365	4.05	1.3987	7.80	2.0541
1.45	0.3716	4.10	1.4110	8.00	2.0794
1.50	0.4055	4.15	1.4231	8.25	2.1102
1.55	0.4383	4.20	1.4351	8.50	2.1401
1.60	0.4700	4.25	1.4469	8.75	2.1691
1.65	0.5008	4.30	1.4586	9.00	2.1972
1.70	0.5306	4.35	1.4701	9.25	2.2246
1.75	0.5596	4.40	1.4816	9.50	2.2513
1.80	0.5878	4.45	1.4929	9.75	2.2773
1.85	0.6152	4.50	1.5041	10.00	2.3026
1.90	0.6419	4.55	1.5151	11.00	2.3979
1.95	0.6678	4.60	1.5261	12.00	2.4849
2.00	0.6931	4.65	1.5369	13.00	2.5649
2.05	0.7178	4.70	1.5476	14.00	2.6391
2.10	0.7419	4.75	1.5581	15.00	2.7081
2.15	0.7655	4.80	1.5686	16.00	2.7726
2.20	0.7885	4.85	1.5790	17.00	2.8332
2.25	0.8109	4.90	1.5892	18.00	2.8904
2.30	0.8329	4.95	1.5994	19.00	2.9444
2.35	0.8544	5.00	1.6094	20.00	2.9957
2.40	0.8755	5.05	1.6194	21.00	3 0445
2.45	0.8961	5.10	1.6292	22.00	3.0910
2.50	0.9163	5.15	1.6390	23.00	3.1355
2.55	0.9361	5.20	1.6487	24.00	3.1781
2.60	0.9555	5.25	1.6582	25.00	3.2189
2.65	0.9746	5.30	1.6677	26.00	3.2581
2.70	0.9933	5.35	1.6771	27.00	3.2958
2.75	1.0116	5.40	1.6864	28.00	3.3322
2.80	1.0296	5.45	1.6956	29.00	3.3673
2.85	1.0473	5.50	1.7047	30.00	3.4012
2.90	1.0647	5.55	1.7138	31.00	3.4340
2.95	1.0818	5.60	1.7228	32.00	3.4657
3.00	1.0986	5.65	1.7317	33.00	3.4965
3.05	1.1154	5.70	1.7405	34.00	3.5264
3.10	1.1314	5.75	1.7492	35.00	3.5553
3.15	1.1474	5.80	1.7579	40.00	3.6889
3.20	1.1632	5.85	1.7664	45.00	3.8066
3.25	1.1787	5.90	1.7750	50.00	3.9120
3.30	1.1939	5.95	1.7834	60.00	4.0943
3.35	1.2090	6.00	1.7918	70.00	4.2485
3.40	1.2238	6. 0	1.8083	80.00	4.3820
3.45	1.2384	6.20	1.8245	90.00	4.4998
3.50	1.2528	6.30	1.8405	100.00	4.6052
3.55	1.2669	6.40	1.8563		
3.60	1.2809	6.50	1.8718		

It will be seen in (8) that the maximum lateral pressure in a bin which must be used in the design of deep bins, is independent of k, and that therefore an exact determination of k is not very important. In calculating the values of V and L in (7) and (8), it is necessary to use a table of

natural or hyperbolic logarithms. A brief table of hyperbolic logarithms is given in Table IV. To find the hyperbolic logarithm of any number, using a table of Brigg's or common logarithms, use the relation: *The hyperbolic or Naperian logarithm of any number = common or Brigg's logarithm* × 2.30259.

The author has calculated the lateral pressures on steel plate bins, Fig. 2.

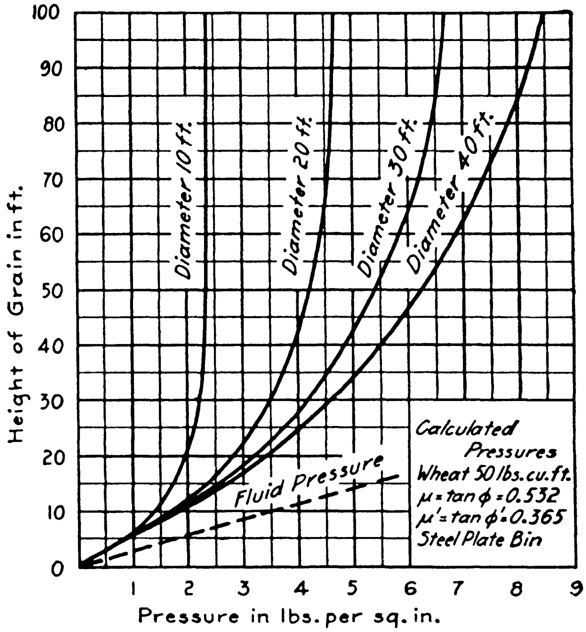


FIG. 2. LATERAL PRESSURE IN STEEL PLATE GRAIN BINS CALCULATED BY JANSSEN'S FORMULA.

To use Fig. 2 to calculate the pressures in rectangular bins, calculate the pressure in a circular or square bin which has the same hydraulic radius, R (R = area of bin ÷ perimeter of bin), as the rectangular bin.

It will be seen in Fig. 2 that the pressure varies as the diameters, where the height divided by the diameter is a constant. By using this principle the pressure for any other diameter within the limits of the diagram may be directly interpolated.

Problem 1. Required the lateral pressure at the bottom of a cement lined bin, 10 ft. in diameter and 20 ft. high, containing wheat weighing 50 lb. per cu. ft. Assume $\mu' = 0.416$, and $k = 0.6$, also R will $= 2\frac{1}{2}$ ft., $w = 50$ lb., $h = 20$ ft., and $k \cdot \mu' = 0.25$.

Now from (8)

$$L = \frac{50 \times 2.5}{0.416}\left(1 - e^{-0.25 \times 20/2.5}\right)$$

$$= 300(1 - e^{-2})$$

Now from Table IV the number whose hyperbolic logarithm is 2.00 is 7.40, and

$$L = 300\left(1 - \frac{1}{7.40}\right),$$

$$= 260 \text{ lb. per sq. ft.},$$

$$= 1.8 \text{ lb. per sq. in.}$$

German Practice.—Janssen's formula is given in Hutte Des Ingenieurs Taschenbuch, as the standard formula for the design of grain bins. For wheat Janssen found that $\mu' = 0.3$, and $k = 0.67$, so that $\mu' \cdot k = 0.20$. Using these values and changing to English units, we have for wheat,

$$V = \frac{w \cdot R}{0.2}(1 - e^{-0.2h/R})$$

or if d = diameter or side of bin, then

$$V = \tfrac{5}{4} w \cdot d(1 - e^{-0.8h/d})$$

$$L = k \cdot V$$

which is the German practice.

Load on Bin Walls.—The walls of a deep bin carry the greater part of the weight of the contents of the bin. The total weight carried by the bin walls is equal to the total pressure, P, of the grain on the bin walls, multiplied by the coefficient of friction μ' of the grain on the bin walls.

From formula (8) the unit pressure on a unit at a depth y will be

$$L = \frac{w \cdot R}{\mu'}(1 - e^{-k \cdot \mu' \cdot y/R}) \tag{10}$$

and the total lateral pressure for a depth y, per unit of length of the perimeter of the bin, will be

$$P = \int_0^y L \cdot dy = \int_0^y \frac{w \cdot R}{\mu'}(1 - e^{-k \cdot \mu' \cdot y/R})dy$$

$$= \frac{w \cdot R}{\mu'}\left[y - \frac{R}{k \cdot \mu'} + \frac{R}{k \cdot \mu'} \cdot e^{-k \cdot \mu' \cdot y/R}\right] \tag{11}$$

Now the last term in (11) is very small and may be neglected for depths of more than two diameters, and

$$P = \frac{w \cdot R}{\mu'}\left[y - \frac{R}{k \cdot \mu'}\right] \text{(approx.)} \tag{12}$$

The total load per lineal foot carried by the side walls of the bin will be

$$P \cdot \mu' = w \cdot R\left[y - \frac{R}{k \cdot \mu'}\right] \text{(approx.)} \tag{13}$$

For the total weight of grain carried by the side walls multiply (13) by the length of the circumference of the bin.

Formulas (12) and (13) may be deduced as follows:—The grain carried by the sides of the bin will be equal to the total weight of grain in the bin minus the pressure on the bottom of the bin. If P is the total side pressure on a section of the bin one unit long, then

$$P \cdot U \cdot \mu' = w \cdot A \cdot y - A \cdot V \tag{a}$$

$$= w \cdot A \cdot y - \frac{w \cdot A \cdot R}{k \cdot \mu'}(1 - e^{-k \cdot \mu' \cdot y/R}) \tag{b}$$

and solving (b)

$$P = \frac{w \cdot A}{\mu' \cdot U}\left[y - \frac{R}{k \cdot \mu'}(1 - e^{-k \cdot \mu' \cdot y/R})\right] \tag{c}$$

$$= \frac{w \cdot R}{\mu'}\left[y - \frac{R}{k \cdot \mu'}(1 - e^{-k \cdot \mu' \cdot y/R})\right] \tag{11}$$

$$= \frac{w \cdot R}{\mu'}\left[y - \frac{R}{k \cdot \mu'}\right] \text{(approx.)} \tag{13}$$

and the total load carried on a section of the bin one unit long will be found by multiplying P in (11) by μ', and

$$P \cdot \mu' = w \cdot R \left[y - \frac{R}{k \cdot \mu'} (1 - e^{-k.\mu'.y/R}) \right]$$

$$= w \cdot R \left[y - \frac{R}{k \cdot \mu'} \right] \text{(approx.)} \quad (13)$$

For example take a steel bin 10 ft. in diameter and 100 ft. deep; weight of wheat, $w = 50$ lb. per cu. ft.; angle of friction of wheat on steel, $\mu' = 0.375$; angle of repose of grain on grain, $\mu = \tan 28° = 0.532$ (μ does not occur in formula (13) but may be used in calculating an approximate value of $k = (1 - \sin 28°)/(1 + \sin 28°) = 0.37$ which is a close approximation to $k = 0.4$ which will be used). Then the load carried by the side walls per lineal foot will be from (13)

$$P \cdot \mu' = 50 \times 2.5 \left[100 - \frac{2.5}{0.4 \times 0.375} \right]$$
$$= 10{,}416 \text{ lb.}$$

The total load on the entire bin walls will be

$$P \cdot \mu' \times 31.416 = 327{,}635 \text{ lb.}$$

The total weight of wheat in the bin is

$$50 \times 78.5 \times 100 = 392{,}700 \text{ lb.}$$

and the total load carried by the bottom of the bin is

$$392{,}700 - 327{,}635 = 65{,}065 \text{ lb.}$$

and the pressure on the bottom $= V = 65{,}065/78.54 = 830$ lb. per sq. ft. From formula (7) we find that $V = 830$ lb. per sq. ft.

EXPERIMENTS ON THE PRESSURE OF GRAIN IN DEEP BINS.—The laws of pressure of grain and similar materials are very different from the well known laws of fluid pressure. Dry wheat and corn come very nearly filling the definition of a granular mass assumed by Rankine in deducing his formulas for earth pressures. As stored in a bin the grain mass is limited by the bin walls, and Rankine's retaining wall formulas are not directly applicable.

If grain is allowed to run from a spout onto a floor it will heap up until the slope reaches a certain angle, called the angle of repose of the grain, when the grain will slide down the surface of the cone. If a hole be cut in the bottom of the side of a bin, the grain will flow out until the opening is blocked by the outflowing grain. There is no tendency for the grain to spout up as in the case of fluids. If grain be allowed to flow from an orifice it flows at a constant rate, which is independent of the head and varies as the diameter of the orifice.

Experiments by Willis Whited,* and by the author at the University of Illinois, with wheat have shown that the flow from an orifice is independent of the head and varies as the cube of the diameter of the orifice. This phenomenon can be explained as follows: The wheat grains in the bin tend to form a dome which supports the weight above. The surface of this dome is actually the surface of rupture. When the orifice is opened the grain flows out of the space below the dome and the space is filled up by grains dropping from the top of the dome. As these grains drop others take their place in the dome. Experiments with glass bins show that the grain from the center of the bin is discharged first, this drops through the top of the dome, while the grain in the lower part of the dome discharges last.

The law of grain pressures has been studied experimentally by several engineers within recent years. A brief resume of the most important experiments is given in the author's "The Design of Walls, Bins and Grain Elevators," where after a careful study of all available experiments the author reached the following conclusions:—

1. The pressure of grain on bin walls and bottoms follows a law (which for convenience will be called the law of "semi-fluids"), which is entirely different from the law of the pressure of fluids.

* Proc. Eng. Soc. of West. Penna., April, 1901.

2. The lateral pressure of grain on bin walls is less than the vertical pressure (0.3 to 0.6 of the vertical pressure, depending on the grain, etc.), and increases very little after a depth of 2½ to 3 times the width or diameter of the bin is reached.

3. The ratio of lateral to vertical pressures, k, is not a constant, but varies with different grains and bins. The value of k can only be determined by experiment.

4. The pressure of moving grain is very slightly greater than the pressure of grain at rest (maximum variation for ordinary conditions is, probably, 10 per cent).

5. Discharge gates in bins should be located at or near the center of the bin.

6. If the discharge gates are located in the sides of the bins, the lateral pressure due to moving grain is decreased near the discharge gate and is materially increased on the side opposite the gate (for common conditions this increased pressure may be two to four times the lateral pressure of grain at rest).

7. Tie rods decrease the flow but do not materially affect the pressure.

8. The maximum lateral pressures occur immediately after filling, and are slightly greater in a bin filled rapidly than in a bin filled slowly. Maximum lateral pressures occur in deep bins during filling.

9. The calculated pressures by either Janssen's or Airy's formulas agree very closely with actual pressures.

10. The unit pressures determined on small surfaces agree very closely with unit pressures on large surfaces.

11. Grain bins designed by the fluid theory are in many cases unsafe as no provision is made for the side walls to carry the weight of the grain, and the walls are crippled.

12. Calculation of the strength of wooden bins that have been in successful operation shows that the fluid theory is untenable, while steel bins designed according to the fluid theory have failed by crippling the side plates.

RECTANGULAR STEEL BINS.—For the calculation of the stresses in and the design of rectangular steel bins, see the author's "The Design of Walls, Bins and Grain Elevators," Second Edition.

CIRCULAR STEEL BINS.—In the designing of steel grain bins particular attention should be given to the horizontal joints, and to the strength of the bin to act as a column to support the grain. To calculate the thickness of the metal the horizontal pressure L is obtained from Janssen's formula, and then the thickness may be found by the formula

$$t = \frac{L \cdot d}{2S \cdot f} \tag{14}$$

where t = thickness of the plate in in.;
L = horizontal pressure in lb. per sq. in.;
d = diameter of bin in in.;
S = working stress in steel in lb. per sq. in.;
f = efficiency of the joint.

The unit stress S may be taken at 16,000 lb. per sq. in., and f will be about 57 per cent for a single riveted lap joint, 73 per cent for a double riveted lap joint, and 80 per cent for double riveted double strap butt joints. For the efficiency of riveted joints, see Table IIa, Chapter XI.

The allowable stresses given for the design of steel mill buildings should be used in design. These allowable stresses are as follows: Tension on net section 16,000 lb. per sq. in.; shear on cross-section of rivets 11,000 lb. per sq. in.; bearing on the projection of rivets (diameter × thickness of plate) 22,000 lb. per sq. in. Compression in columns $P = 16{,}000 - 70l/r$ where P = unit stress in lb. per sq. in., l = length of member and r = radius of gyration of the member, both in inches.

Rivets in Horizontal Joints.—The side walls carry a large part of the weight of the grain in the bin and this should be considered in designing the horizontal joints. The weight of the grain supported by the bin above any horizontal joint can be calculated as shown in the following example. Assume a steel plate bin 25 ft. in diameter, and it is required to calculate the grain

supported by the bin walls above a horizontal joint 75 ft. below the top of the grain. From equation (13) the grain carried by the bin walls per lineal foot of circumference of bin, where w = 50 lb. per cu. ft.; $\mu' = 0.375$; $k = 0.40$, also $R = 25/4 = 6.25$, and

$$P\cdot\mu' = 50 \times 6.25\left[75 - \frac{6.25}{0.4 \times 0.375}\right]$$
$$= 10{,}415 \text{ lb.}$$

The weight of the steel bin above the joint may be taken as 1,250 lb. per lineal foot of joint. The horizontal riveting should then be designed for a shear of 11,665 lb. per lineal foot of joint. Assume that the plates are ⅜ in. thick and the rivets ¾ in. in diameter. For allowable stresses of 16,000 lb. per sq. in. in tension, 11,000 lb. per sq. in. in shear, and 22,000 lb. per sq. in. in compression; then, Table 114, Part II, the value of a ¾ in. shop rivet in single shear = 4,860 lb., and a field rivet is ⅔ of 4,860 = 3,240 lb., and in compression = 6,190 lb. for shop rivets and = 4.127 lb. for field rivets. For a lap joint therefore the spacing should not be greater than 3,240 × 12 ÷ 11,665 = 3.25 in., requiring but one row of rivets.

Stresses in a Steel Bin Due to Wind Moment.—If M is the moment due to the wind acting on the bin above the horizontal joint, then the stress per lineal foot of joint due to wind moment will be

$$S = \frac{M\cdot d}{2I}, \text{ but } I = \tfrac{1}{8}\pi\cdot d^3 \text{ (approx.) and } S = \frac{4M}{\pi\cdot d^2} \tag{15}$$

where all dimensions are in feet. For a wind load of 30 lb. per sq. ft. on two-thirds of the tank (20 lb. per sq. ft. on the entire surface of the tank) the wind stress will be S = 2,865 lb. per lineal foot. The spacing therefore should not be greater than 3,240 × 12 ÷ (11,665 + 2,865) = 2⅝ in.

Stiffeners.—In large circular steel bins the thin side walls are not sufficiently rigid to support the weight of the grain and it is necessary to supply stiffeners. For this purpose angles or Z-bars may be used. Experience has shown that bins in which the height is equal to or greater than about 2½ times the diameter do not need stiffeners. There is at present no rational method for the design of these stiffeners or the stiffeners in plate girders. In Fig. 9 will be seen the details of a steel bin of the Independent Steel Elevator with Z-bar stiffeners. Angle stiffeners were used in the bins of the Electric Elevator, Minneapolis, Minn.

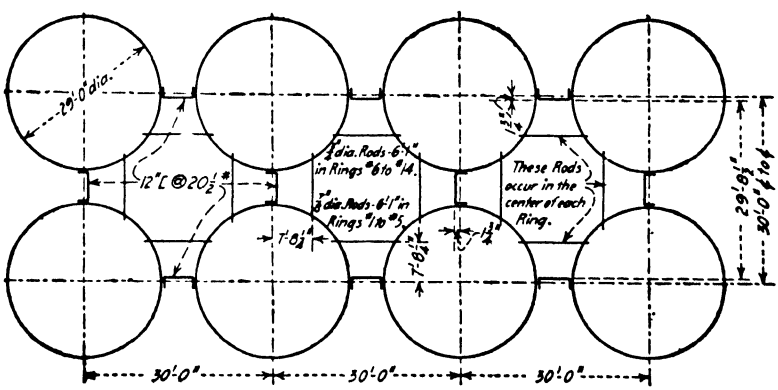


FIG. 3. PLAN OF STEEL STORAGE BINS FOR A STEEL ELEVATOR.

Circular steel bins are used for storage in large elevators and may be used for a complete elevator as in Fig. 3. The space between the bins is sometimes used for auxiliary storage. The circular bin walls are stiffened by means of vertical channels, and the auxiliary bins are cross-braced with steel rods. Complete details of circular steel bins for the Independent Elevator, Omaha, Neb., are shown in Fig. 9.

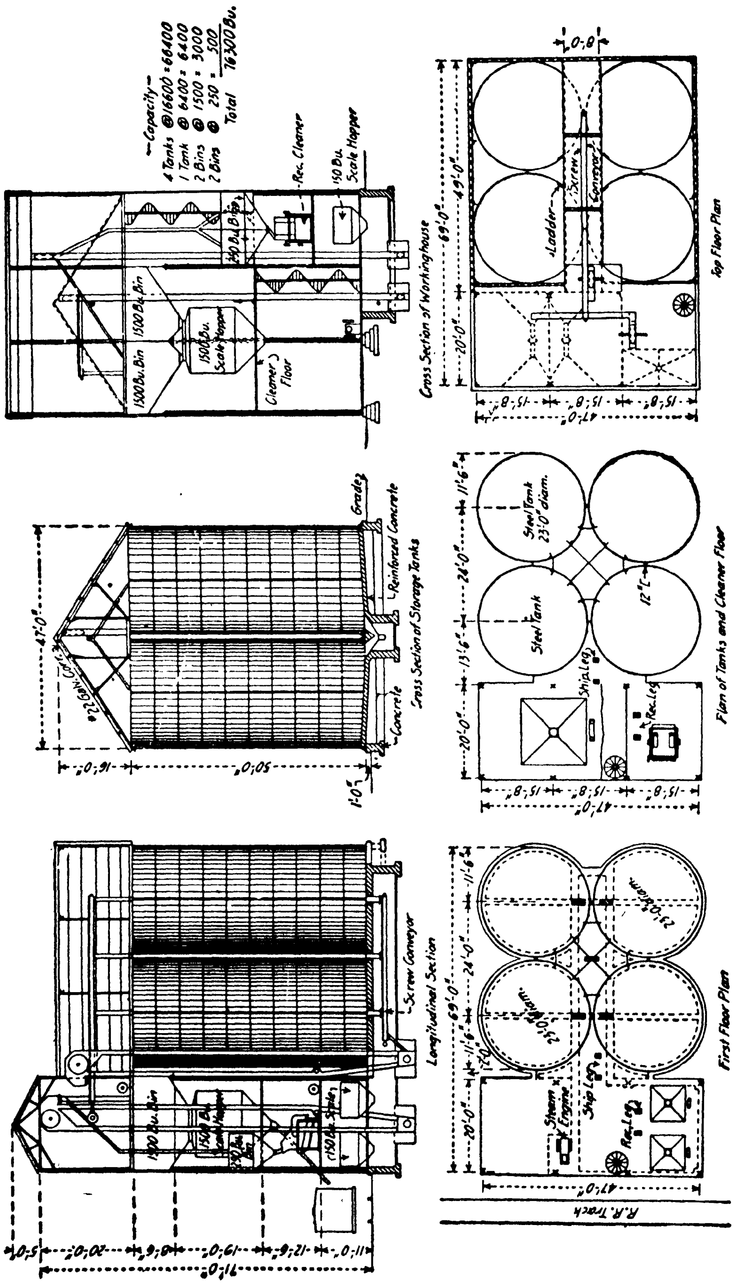


FIG. 4. STEEL ELEVATOR FOR MANHATTAN MILLING CO., MANHATTAN, MONTANA.

Steel Country Elevator.—General plans of a steel grain elevator for the Manhattan Milling Co., designed and constructed by the Minneapolis Steel & Machinery Co., Minneapolis, Minn., are given in Fig. 4. This elevator could easily be changed to a shipping elevator by putting in a wagon dump. Grain is run from the cars into the boot of the receiving leg, and is then elevated and conveyed by a screw conveyor to the large storage bins, or is run into the temporary storage bins, then cleaned and elevated and conveyed to the storage bins by the screw conveyor. The bins are built of steel plates, and the working house is built of steel framework covered with corrugated steel. This elevator has a capacity of 76,300 bushels but the scheme can be used for a 30,000 to 40,000 bushel elevator for either shipping or for milling purposes.

THE INDEPENDENT STEEL ELEVATOR, OMAHA, NEB. General Description.—This elevator consists of a steel working house having a bin capacity of 240,000 bushels and 8 steel storage bins having a storage capacity of 100,000 bushels each, making a total storage capacity of 1,040,000 bushels.

The steel working house is 64 ft. × 70 ft., with 14 ft. sheds on two ends and one side, as shown in Fig. 5. The sub-story of the building is 26 ft. The bins are 64 ft. 4 in. high, as shown in Fig. 6, and are supported on steel columns, as shown in Fig. 6 and Fig. 7. The spouting story is 24 ft. 6 in. high; the garner and scale story is 26 ft. 6 in. high; and the machinery story is 13 ft. 8 in. high. The walls below and above the bins are covered with No. 24 corrugated steel laid with 1½ corrugations side lap and 3 in. end lap. The roof is covered with No. 22 corrugated steel laid directly on the steel purlins with 2 corrugations side lap and 6 in. end lap.

On the first or working floor the floor between the tracks is made of ¼ in. plate bolted to the beams, while the remainder of this floor is made of concrete filled in above concrete arches which rest on the flanges of the beams with a finish 1¼ in. thick of Portland cement mortar consisting of one part cement to one part clean, sharp sand. The concrete is composed of one part Portland cement, two parts sand, and five parts crushed stone.

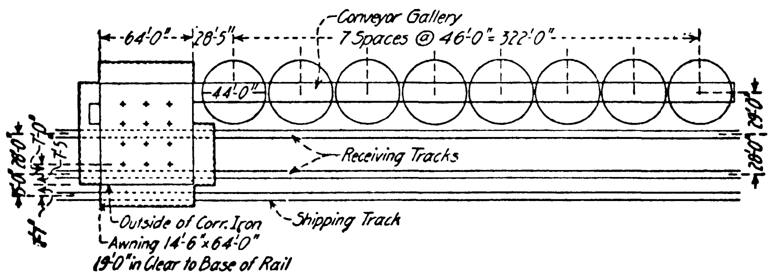


FIG. 5. PLAN OF INDEPENDENT ELEVATOR.

The floor of the cupola throughout the different floors and in the gallery leading over the bins is made of No. 24 corrugated steel resting on steel framework, and covered with 3 in. of concrete and a one-inch finish of one to one Portland cement mortar troweled smooth. All doors are of the rolling steel type. The window frames were made of 2 in. × 6 in. timbers and are covered with No. 26 sheet steel. All windows are provided with 1¾ in. checked rail sash and are glazed with double strength glass.

Painting.—All steel work of every description was painted with one coat oxide of iron paint at the shop and a second coat after erection. The tank plates and corrugated steel were painted on the exterior surface only after erection.

Bins.—The eight steel storage bins are 44 ft. in diameter and 80 ft. high, have a capacity of 100,000 bushels and rest on separate concrete foundations. The bins are constructed of steel plates stiffened with Z-bars, as shown in Fig. 9. The bins are covered with a steel plate roof, Fig. 12, supported on roof trusses, as shown in Fig. 11 and Fig. 13. A conveyor gallery 10 ft.

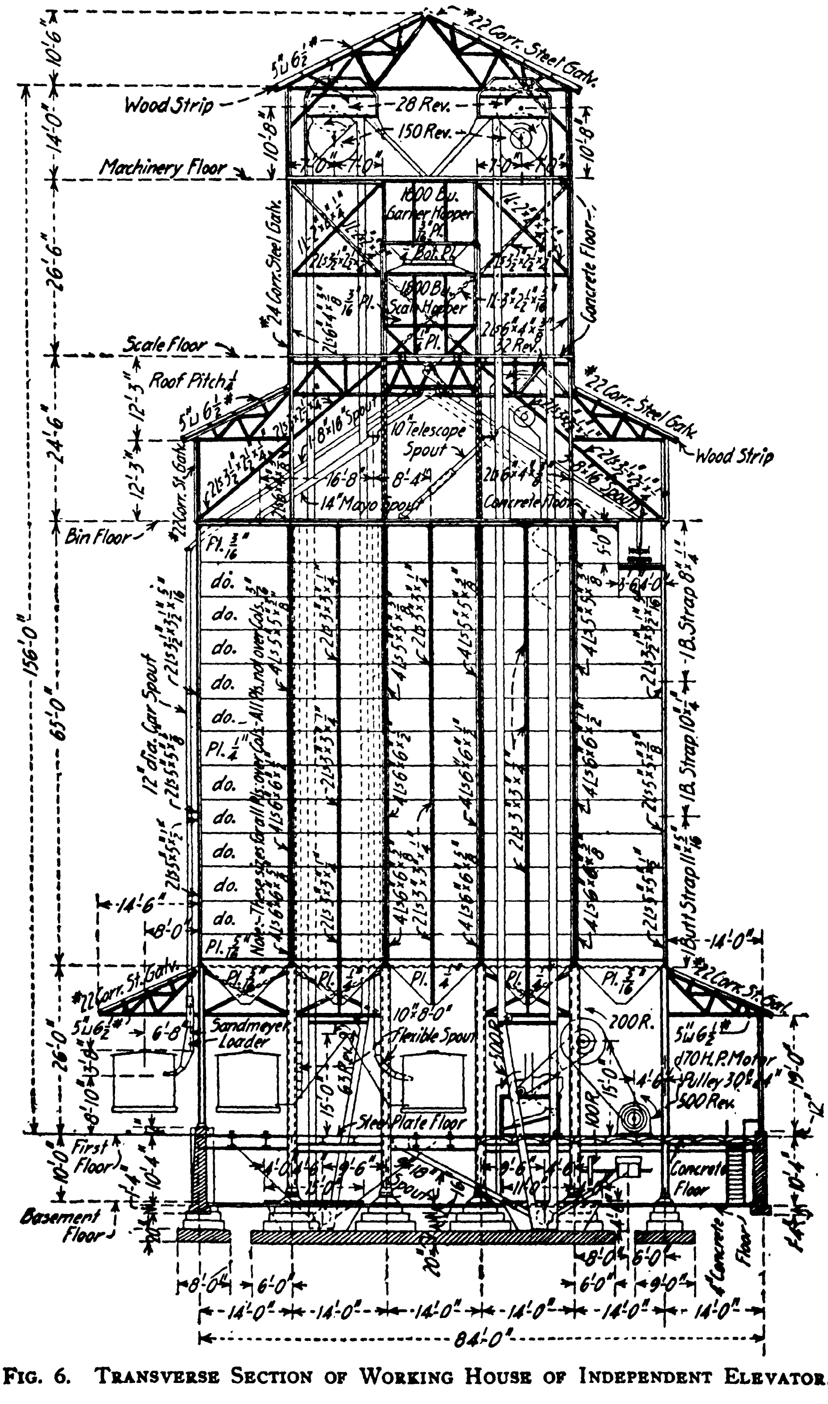


FIG. 6. TRANSVERSE SECTION OF WORKING HOUSE OF INDEPENDENT ELEVATOR.

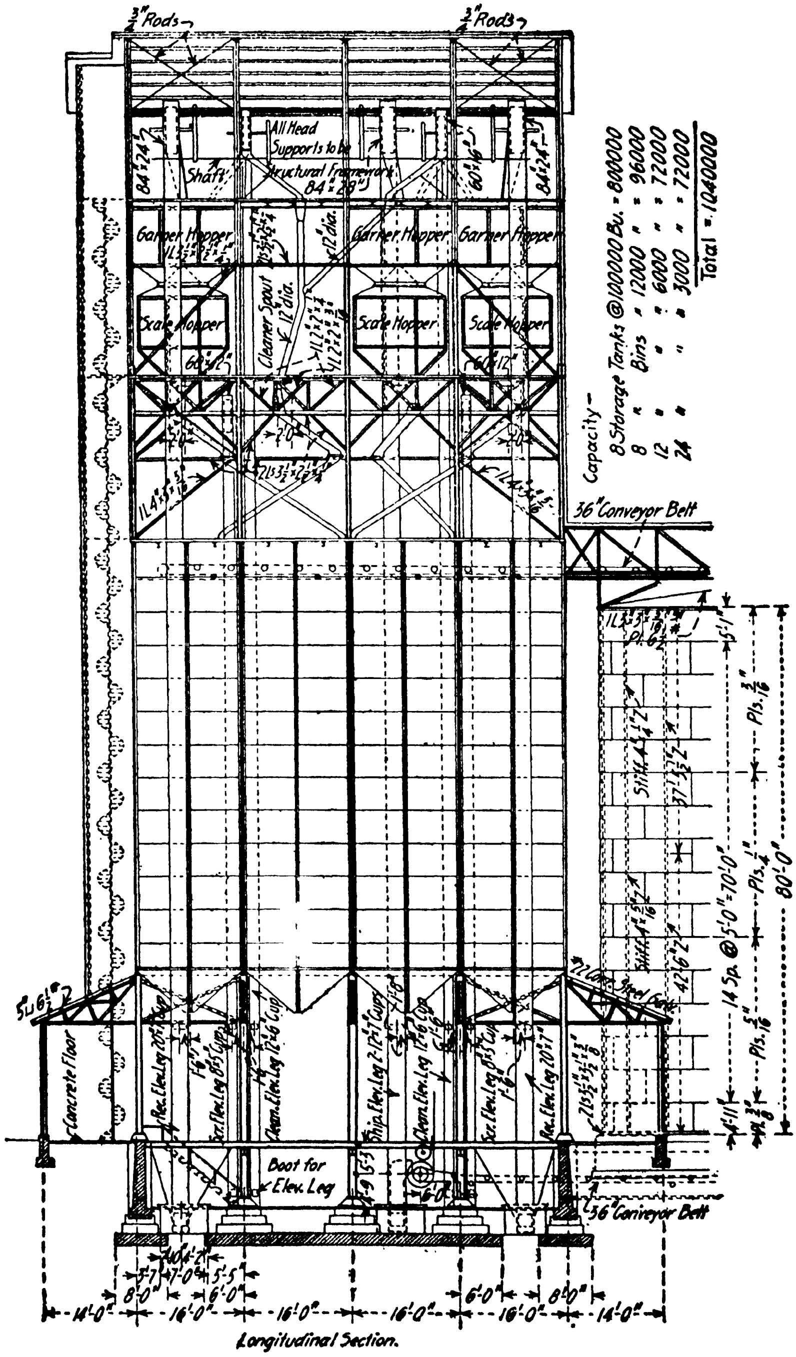


FIG. 7. LONGITUDINAL SECTION OF WORKING HOUSE OF INDEPENDENT ELEVATOR.

wide and 8 ft. high extends from the working house over the bins. A conveyor tunnel extends from the working house under the bins. The rivet spacing in the circular bins is shown in Fig. 9.

The bins in the working house are arranged as shown in Fig. 8, and are constructed of plates, as shown in Fig. 6 and Fig. 7. The bins, 14 ft. × 16 ft., are braced in the corners with angle braces spaced 5 ft. centers vertically, and of the sizes shown in Fig. 8. The large bins are also braced with ⅞ and ¾-in. round rods spaced 5 ft. apart as shown. All the smaller bins are braced with ⅝-in. round rods spaced 2 ft. 6 in. apart as shown. Vertical angles in the sides of the bins are provided, as shown in Fig. 6, Fig. 7, and Fig. 8.

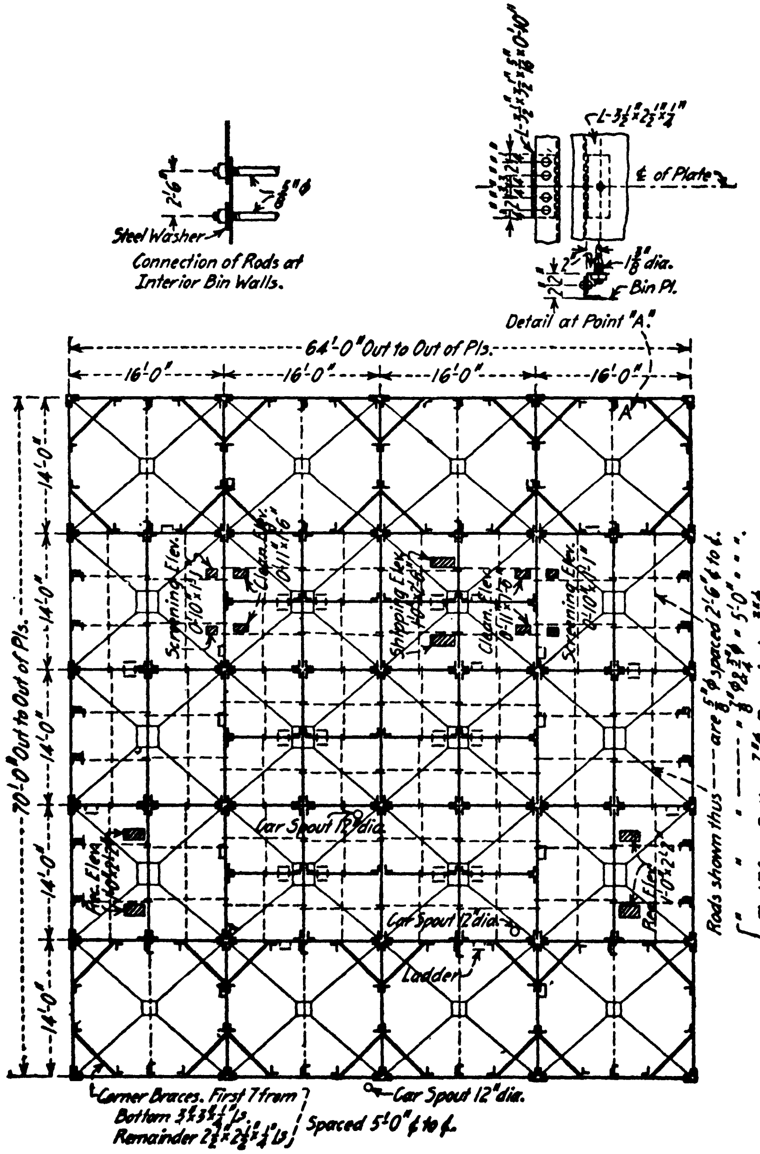


FIG. 8. PLAN OF BINS IN WORKING HOUSE OF INDEPENDENT ELEVATOR.

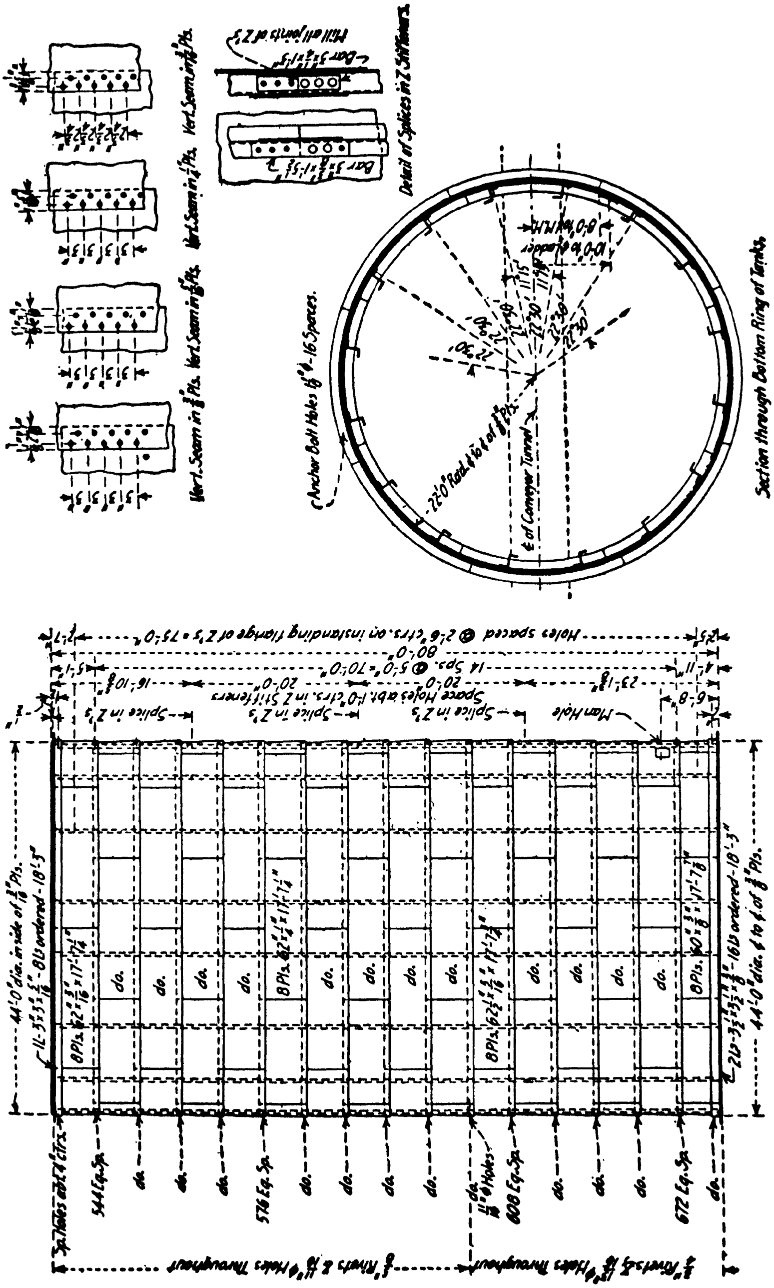


FIG. 9. DETAILS OF STEEL BINS FOR INDEPENDENT ELEVATOR.

EQUIPMENT.—There are two stands of receiving elevators with receiving pits on either side. These elevators have 22-inch 6-ply belts and 20 in. × 7 in. × 7 in. buckets spaced 14 in. apart; the receiving pits are covered with steel grating, and a pair of Clark's automatic grain shovels are located at each unloading place. These elevators are driven with an electric motor of 100 H. P., each elevator being driven with a clutch and pinion so that the elevator may be stopped and started at will.

There is one stand of shipping elevators constructed in the same manner, having a 26-in. 6-ply belt and 2 lines of 12 in. × 7 in. × 7 in. buckets spaced 14 in. apart.

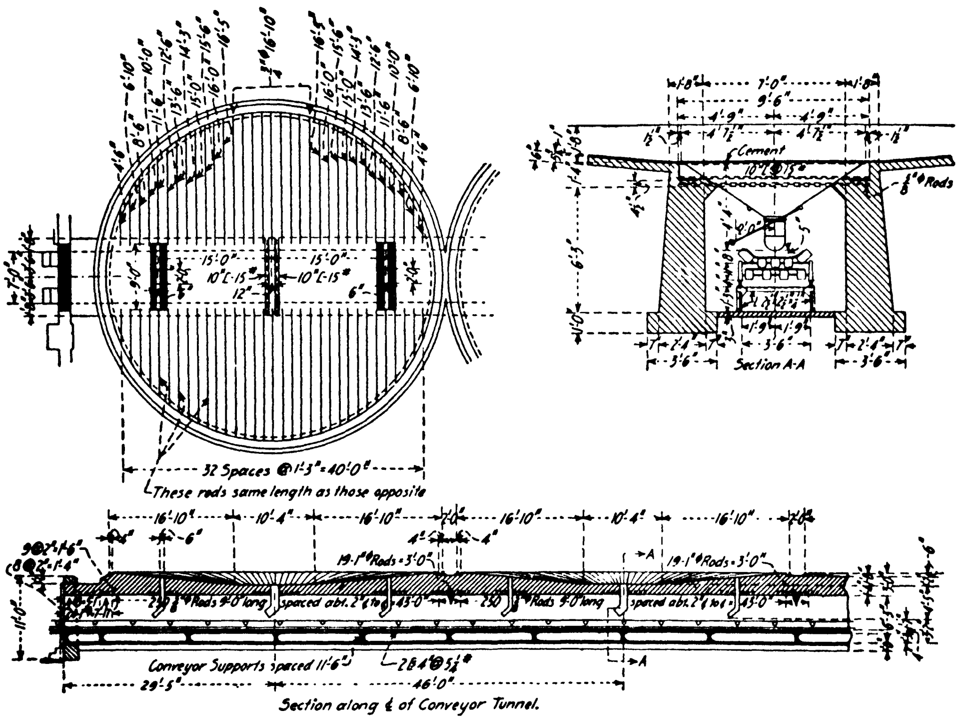


FIG. 10. DETAILS OF BIN BOTTOMS AND CONVEYORS UNDER BINS, INDEPENDENT ELEVATOR.

There are two stands of cleaning elevators with 14-in. 6-ply belts with 12 in. × 6 in. × 6 in buckets spaced 12 in. apart.

There are also two screenings elevators with 9-in. 5-ply belts with 8 in. × 5 in. × 5 in. buckets spaced 12 in. apart.

The shipping, screenings, and cleaner elevators are driven from a line shaft which is driven by a 100 H. P. motor, each elevator being driven by a core wheel and pinion.

Three scale hoppers, having a capacity of 1,800 bushels, are located in the cupola, and three garner hoppers of 1,800 bushels capacity are located above the scale hoppers.

The main line shaft on the first floor is driven by a 170 H. P. motor.

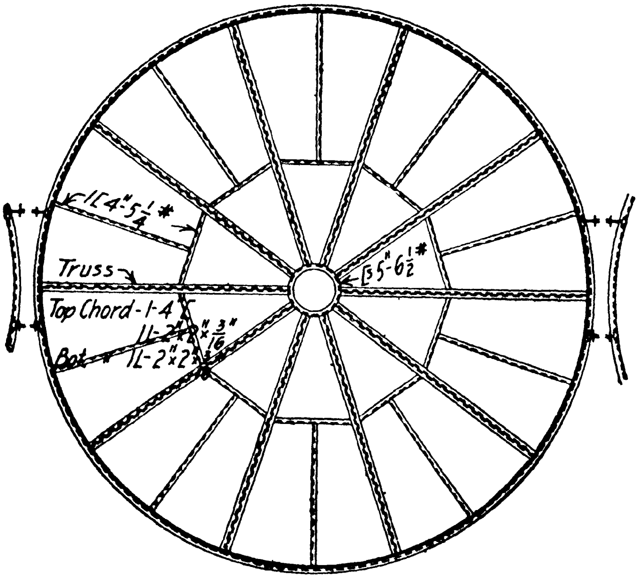
A car puller capable of moving 25 loaded cars is provided.

Elevators.—The boots of the receiving and shipping elevators rest in water-tight steel boot tanks made of $\frac{5}{16}$-in. steel plates. The elevator boots are made of $\frac{5}{16}$-in. steel plates, the boot pul

leys having a vertical adjustment of 8 inches. The elevator cases are made of No. 12 steel up to the bins, and of $\frac{3}{16}$-in. plates in the bins, and No. 14 steel above the bins. The cases are strengthened by angles at the corners. The elevator heads are made of No. 14 steel. At each receiving elevator is a large elevator pit extending from the leg back to the center of the track. This pit is constructed of beams and $\frac{3}{16}$-in. plates and is covered with a grating of $1\frac{3}{4} \times \frac{1}{4}$-in. bars spaced $1\frac{1}{4}$ in. apart.

The elevator buckets are "Buffalo" buckets; those for the receiving elevators are 20 in. × 7 in. × 7 in.; for the shipping elevators two lines of 12 in. × 7 in. × 7 in. buckets; for the cleaning elevators one line of 12 in. × 6 in. × 6 in. buckets; and for the screenings elevator one line of 8 in. × 5 in. × 5 in. buckets. The buckets in the receiving, shipping and cleaning elevators are spaced 14 in. apart, while those in the screenings elevator are spaced 12 in. apart.

The elevator belts in the receiving elevators are 22 in. wide and 6-ply, the shipping belts are 26 in. wide and 6-ply; the cleaning belts are 14 in. wide and 6-ply, and the screenings belts are 9 in. wide and 5-ply. The belting is made of 32 ounce duck and is first-class.



Roof Framing Plan for Tanks.

FIG. 11. FRAMING FOR ROOF OF CIRCULAR BINS, INDEPENDENT ELEVATOR.

Spouts.—The building is provided with a complete system of spouts. The general distributing spouts from the scales to the shipping spouts are double-jointed Mayo spouts. There are three shipping spouts which are provided with telescoping bottom sections. All bin bottoms are provided with a revolving spout with a cut-off gate operated with a rack and pinion, with cords leading to within reaching distance of the floor.

Conveyors.—The conveyor belt leading from the working house over the bins is a 36 in. 4-ply conveyor belt, is carried on disc rolls consisting of 3 straight-faced 6-in. pulleys and 2 special discs; the discs run loose on the shafts, which are $1\frac{3}{16}$-in. diameter and are spaced 5 ft. centers. The return rolls are 5-in. straight-faced rolls spaced 15 ft. centers. At each point in the elevator where grain is loaded onto the belt there are two pairs of special concentrating rolls. Movable

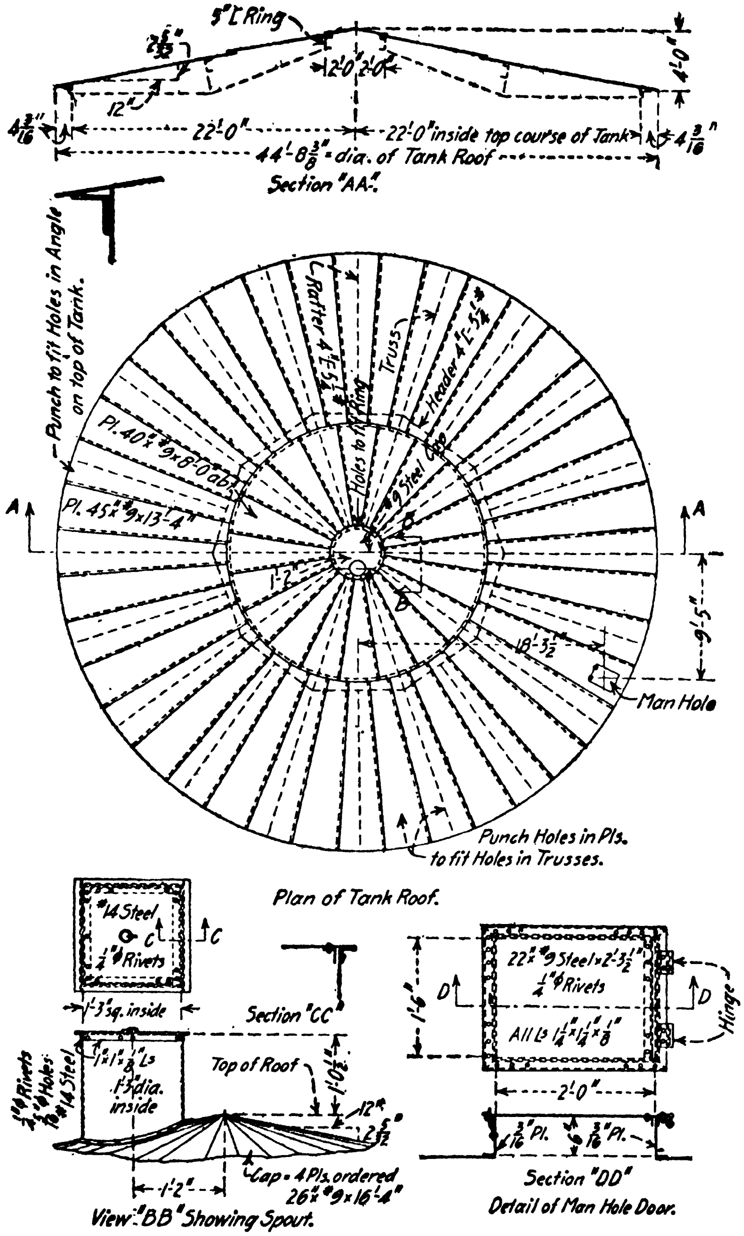


FIG. 12. DETAILS OF STEEL ROOF FOR STEEL BINS FOR INDEPENDENT ELEVATOR.

trippers provided with spouts are provided, so that grain may be discharged on either side of the belt. The entire conveyor is carried on a steel framework. The conveyor belt is driven by a 40 H. P. motor. The conveyor in the tunnel leading from the storage tanks to the working house is of the same type as the conveyor above the bins, and is supported on a steel framework, except that the top or carrying rolls are all of the concentrating types, as shown in Fig. 10. The concentrating rollers are composed of two straight-faced rolls from the main shaft, and two concentrating rolls meeting at an angle of 45° to the straight rolls. The lower conveyor is driven by a rope drive from the main line shaft in the working house.

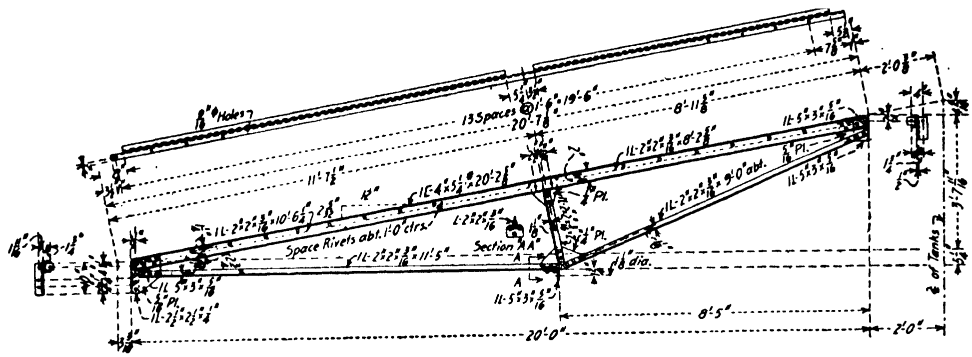


FIG. 13. DETAILS OF STEEL ROOF TRUSS FOR STEEL BINS, INDEPENDENT ELEVATOR.

Scale Hoppers.—There are three scale hoppers of 1,800 bushels capacity, each mounted on a Fairbanks-Morse and Company's scales, having a capacity of 84,000 lb., and have steel frames. The hoppers have $\frac{3}{16}$-in. steel plate sides, and $\frac{1}{4}$-in. plate bottoms, stiffened with angle irons, and are tied together with tie rods. Each hopper is provided with a 22-in. cast iron outlet with a steel plate cut-off gate.

Garners.—A steel garner hopper is placed directly over each scale hopper. The garners have a capacity of 1,800 bushels, and are constructed with $\frac{3}{16}$-in. side plates and $\frac{1}{4}$-in. bottom plates. The bottoms of the garners are hoppered to four openings, which are provided with gates sliding on steel rollers.

Cleaning Machines.—A large size cleaning machine and a large size oat clipper are provided. These machines are connected with a large dust collector which discharges the dust from the cleaning machines and from the sweepings outside of the building.

Car Puller.—A car puller having a capacity of 25 loaded cars is provided. The car puller has two drums, each provided with 400 ft. of $\frac{5}{8}$-in. crucible steel cable.

Shovels.—A pair of Clark automatic grain shovels, with all necessary counterweights, sheaves, scoops, etc., are provided.

The total weight of steel in the elevator is 1,700 tons; approximately 900 tons in the working house, and 800 tons in the circular bins and conveyors.

The total cost was $205,000, of which the 8 steel bins and conveyors cost $80,000.

COST OF STEEL GRAIN ELEVATORS.—The following costs of steel grain elevators have been taken from the author's "The Design of Walls, Bins and Grain Elevators," which also gives costs of reinforced concrete and tile bins, and timber grain elevators. The total cost of the steel grain elevator of the working house type, constructed by the Great Northern Railway at Superior, Wis., was 39.65 cts. per bushel of storage. The elevator had a storage capacity of 3,100,000 bushels, and the steel weighed 7 lb. per bushel of storage capacity. The Independent

Elevator cost 9½ cts. per bushel storage capacity for the steel bins, and 54 cts. per bushel storage capacity for the working house. A steel country elevator having four steel tanks, 17½ ft. diameter and 30 ft. high, with an interspace bin and a conveyor shed, and having a storage capacity of 30,000 bushels, weighed 3 lb. per bushel of storage capacity. The shop cost and cost of erection of the structural steel was $15.00, and $19.00 per ton, respectively.

References.—For the design of reinforced concrete grain bins and elevators, and for additional data and examples, see the author's "The Design of Walls, Bins and Grain Elevators."

CHAPTER X.

Steel Head Frames and Coal Tipples.

Types of Head Works for Mines.—The design of the head works for a mine depends upon the material which is to be hoisted, upon the depth of the mine, the inclination of the shaft, the rate of hoisting, the amount to be hoisted at one time, the treatment of the ore or coal after being hoisted, and upon the material used in the construction of the structure. Head works for mines may be divided into three classes: (1) head frames; (2) rock houses; (3) coal tipples.

The first head frames were constructed of timber; the most common type being the 4-post head frame. The square or rectangular mine tower was cross-braced and the sheave supports were made of heavy timber. The back brace was inclined and was placed between the hoisting rope and the line of the resultant of the stress in the hoisting rope.

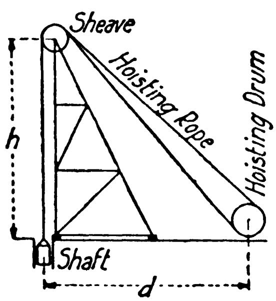


Fig. 1.

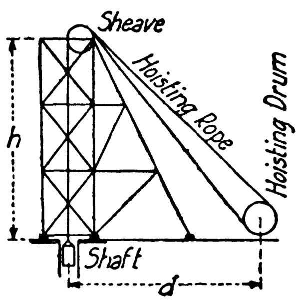


Fig. 2.

Steel head frames vary in design to suit local conditions and the ideas of the designer. The A-frame in Fig. 1 is the most satisfactory type where conditions permit of its use. It is simple in design and economical of material; the stresses are statically determinate, and it can be easily and effectively braced, making a very rigid frame. The 4-post frame in Fig. 2 is the type to use when it is necessary to hoist from several compartments of a shaft not in a single line. It is also used for coal tipples and double compartment shafts. The 4-post frame is not so economical of material as the A-frame; is more difficult to brace effectively, partly for the reason that part of the bracing in the tower must be omitted to permit the dumping of the ore or coal, and in addition the stresses are statically indeterminate. The frame shown in Fig. 3 is a modification of the A-frame used for an inclined shaft. Several early head frames in the coal fields of Pennsylvania were built on the lines of the frame shown in Fig. 4. This type of frame has no points of merit and is practically obsolete.

For an elaborate discussion of the design of head frames, coal tipples, and other mine structures, see the author's "The Design of Mine Structures."

METHODS OF HOISTING.—In hoisting from inclined or vertical shafts, the hoisting engine is placed at some distance from the mouth of the shaft, the cable passes up over the sheave at the top of the head frame and into the shaft. The rope, if round, is carried on a smooth or a grooved hoisting drum, and if flat, is carried on a hoisting reel. The maximum working load on the rope occurs when the loaded skip or cage is being hoisted from the bottom of the shaft. The working load then consists of the skip or cage, the load, the accelerating force, the weight of the

rope itself, and the friction of the rope on the sheave and drum and of the skip or cage in the guides.

With round ropes the hoisting drum for deep mines is commonly made conical, the small diameter being used when the load is at the bottom of the shaft. Flat ropes are wound on a reel,

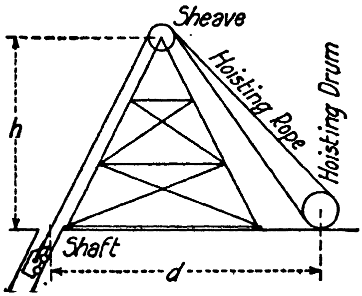


FIG. 3.

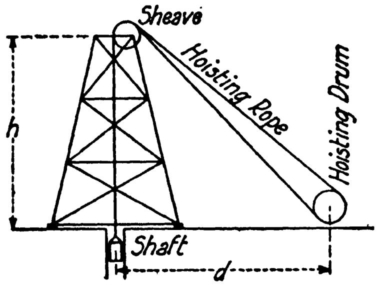


FIG. 4.

so that the small diameter is used when the load is at the bottom of the shaft, the diameter of the reel increasing as the rope is wound up. The required height of the head frame depends upon (1) the room required for screening, crushing and handling the coal or ore; (2) the speed of hoisting—with rapid hoisting it is necessary to have a height from the landing to the sheaves

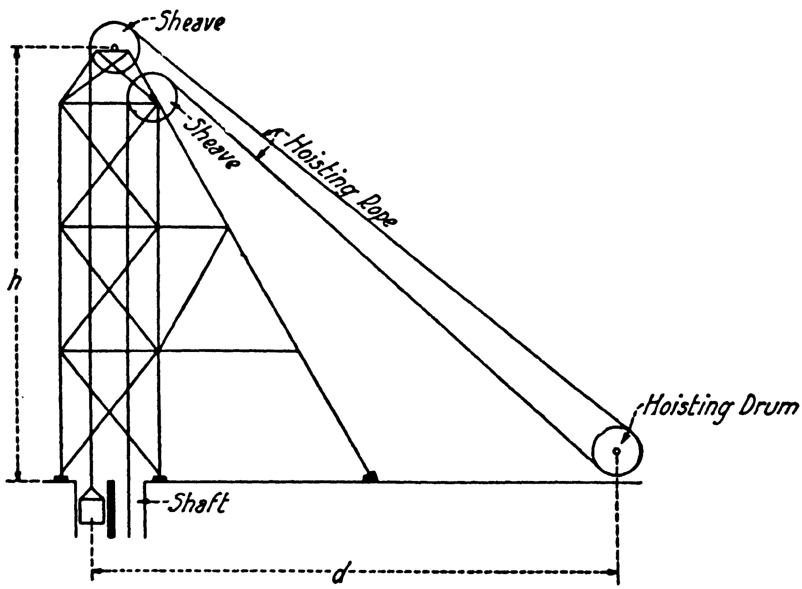


FIG. 5.

of from two to three times the height of the cage or skip or a full revolution of the drum to prevent over winding, and (3) the desired location of the hoisting engine. With a given height of head frame *h*, the distance *d*, Figs. 1 to 5, depends upon the diameter of the sheave, the diameter of the rope, and whether the rope is round or flat. The sheave should be as large as can conveniently

be used, as the larger the sheave the longer the life of the hoisting rope. The inertia of a large, heavy sheave, however, with rapid hoisting may kink the rope and cause excessive wear. The bending stresses in flat ropes for a sheave of given diameter are less than in round ropes having equal strength, but the life of flat ropes is less than for round ropes. Flat ropes are wound on reels which are at all times in line with the head frame sheave, while round ropes are wound on a drum so that the horizontal angle between the center line of the sheave and the cable is continually changing. The distance, d, for flat ropes can then be less than for round ropes.

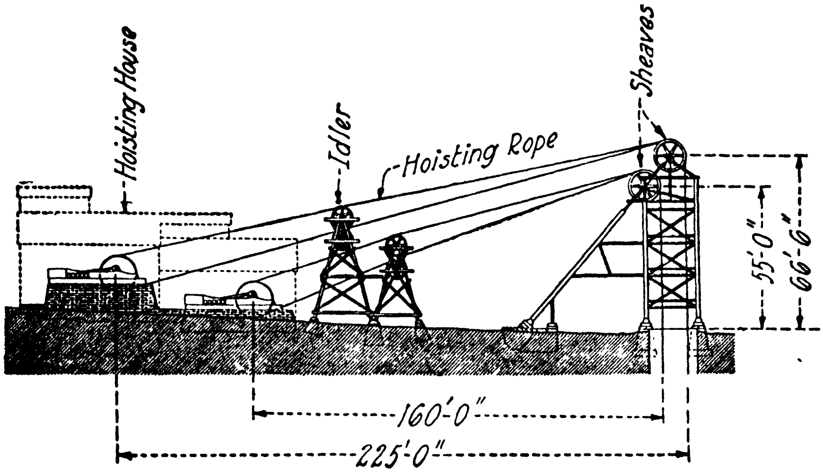


FIG. 6. GILBERTON STEEL HEAD FRAME.

Hoisting from mine shafts is commonly done in two compartments of the shaft at the same time, the unloaded skip or cage descending as the loaded skip or cage ascends. This is known as hoisting in balance or counterbalance. There is a considerable saving in power in hoisting in balance. To hoist in balance it is necessary to take ore from one level with both skips unless the Whiting system is used. When a round rope winds off the drum it makes an angle with the groove in the sheave on the head frame and the friction increases the tension in the cable and also reduces its life. To reduce the friction and wear the hoisting engines are placed at a considerable distance back from the head frame.

The head frame may be placed so that the sheaves are parallel, as in Figs. 1 to 4, or so that the sheaves are in tandem, as in Figs. 5 and 6. With the latter method it is necessary to place the hoisting engine farther from the shaft than where the sheaves are in parallel. Where the hoisting engine is placed well back from the shaft it becomes necessary to support the hoisting rope on idlers, as shown in Fig. 6. Where mines have three compartment shafts, ore is commonly hoisted from but two compartments, the third compartment being used for pumps, pipes, etc. This arrangement makes it necessary to place the head sheaves so that they will not be symmetrical with the center line, bringing heavier working stresses on one side of the head frame than on the other side.

Hoisting from Deep Mines.—In deep mines the rope in the mine becomes a large part of the load and various methods have been used to counterbalance the weight of the rope. Four methods for obviating the difficulty just mentioned have been used: (1) the Koepe system; (2) the Whiting system; (3) modifications of (1) and (2), and (4) by the use of a taper rope. These methods are described in the author's "The Design of Mine Structures."

HOISTING ROPES.—Round hoisting ropes are commonly made of six strands, each of which is formed by twisting nineteen wires together, the strands being wound around a hemp

center. Wire strands are twisted around the core either to the right or the left, and the resulting rope is either "right lay" or "left lay." The twist may be long or short; the shorter twist forms a more flexible rope, while the longer twist forms a more rigid rope. Wire rope is made of iron, open-hearth steel, crucible steel, and plough steel. The strength of the wire from which the rope is made is about as follows: iron wire, 40,000 to 100,000 lb. per sq. in.; open-hearth steel wire, 50,000 to 130,000 lb. per sq. in.; crucible steel wire, 130,000 to 190,000 lb. per sq. in.; and plough steel wire, 190,000 to 350,000 lb. per sq. in. Hoisting ropes are usually made of crucible cast steel or plough steel.

Flat wire rope is composed of several round ropes whose diameter is equal to the required thickness of the flat rope, laid side by side and sewed together with iron or annealed cast steel wire. The round ropes are alternately of right and left lay or twist, have four strands without either hemp or wire center. The number of wires in each strand is usually seven, but may be nineteen. The chief drawbacks to the use of flat wire rope are its first cost and the rapid wear of the sewing wires.

Flat ropes and reels are used to a limited extent in the western part of the United States, while round ropes are generally used in hoisting coal and in the deep copper and iron mines in Michigan.

Strength of Wire Rope.—The dimensions, weight and strength of round crucible steel hoisting rope are given in Table I, while similar data for plough steel hoisting rope are given in Table II. The strengths of wire rope given by the different makers differ somewhat.

TABLE I.

CAST STEEL HOISTING ROPE. ULTIMATE STRENGTH, WORKING STRENGTH AND WEIGHT OF WIRE ROPE COMPOSED OF 6 STRANDS AND A HEMP CENTER, 19 WIRES TO THE STRAND.

Diameter, In.	Approximate Circumference, In.	Weight per Ft., Lb.	Safe Working Load, for Hoisting, L, Lb.	Approximate Breaking Stress, Lb.	Safe Working Stress for Direct Pull, S, Lb.	Minimum Size of Drum or Sheave, Ft
$2\frac{3}{4}$	$8\frac{5}{8}$	11.95	Safe working load, L, $= 2S-$bending stress.	456,000	76 000	10
$2\frac{1}{2}$	$7\frac{7}{8}$	9.85		380 000	66,300	$9\frac{1}{2}$
$2\frac{1}{4}$	$7\frac{1}{8}$	8.00		312,000	52,000	$8\frac{1}{2}$
2	$6\frac{1}{4}$	6.30		248,000	41,300	8
$1\frac{3}{4}$	$5\frac{1}{2}$	4.85		192,000	32,000	$7\frac{1}{4}$
$1\frac{5}{8}$	5	4.15		168,000	28,000	$6\frac{1}{4}$
$1\frac{1}{2}$	$4\frac{3}{4}$	3.55		144,000	24,000	$5\frac{3}{4}$
$1\frac{3}{8}$	$4\frac{1}{4}$	3.00		124,000	20,700	$5\frac{1}{2}$
$1\frac{1}{4}$	4	2.45		100,000	16,700	5
$1\frac{1}{8}$	$3\frac{1}{2}$	2.00		84,000	14,000	$4\frac{1}{2}$
1	3	1.58		68,000	11,300	4
$\frac{7}{8}$	$2\frac{3}{4}$	1.20		52,000	8,700	$3\frac{1}{2}$
$\frac{3}{4}$	$2\frac{1}{4}$	0.89		38,800	6,300	3
$\frac{5}{8}$	2	0.62		27,200	4,500	$2\frac{1}{4}$
$\frac{9}{16}$	$1\frac{3}{4}$	0.50		22,000	3,700	$1\frac{3}{4}$
$\frac{1}{2}$	$1\frac{1}{2}$	0.39		17,600	2,900	$1\frac{1}{2}$
$\frac{7}{16}$	$1\frac{1}{4}$	0.30		13,6-0	2,300	$1\frac{1}{4}$
$\frac{3}{8}$	$1\frac{1}{8}$	0.22		10,000	1,670	1
$\frac{5}{16}$	1	0.15		6,800	1,170	$\frac{3}{4}$
$\frac{1}{4}$	$\frac{3}{4}$	0.10		4,800	800	$\frac{1}{2}$

Working Load on Hoisting Rope.—The stresses in a hoisting rope are the sum of the stresses due to (1) the weight of the rope, (2) the friction of the rope, (3) the bending of the rope over the head sheave, (4) the live load, and (5) the impact due to starting and stopping the load. The stresses due to bending are discussed in the next section. The stresses due to impact vary from zero to twice the working load if the hoisting cable is taut, and to several times the working load

TABLE II.

PLOUGH STEEL HOISTING ROPE. ULTIMATE STRENGTH, WORKING STRENGTH AND WEIGHT OF WIRE ROPE COMPOSED OF 6 STRANDS AND A HEMP CENTER, 19 WIRES TO THE STRAND.

Diameter, In.	Approximate Circumference, In.	Weight per Ft., Lb.	Safe Working Load for Hoisting, L, Lb.	Approximate Breaking Stress, Lb.	Safe Working Stress for Direct Pull, S, Lb.	Minimum Size of Drum or Sheave, Ft.
2 3/4	8 5/8	11.95	Safe working load, L, = $2S$ − bending stress.	550,000	91,700	14
2 1/2	7 7/8	9.85		458,000	76,300	12 1/2
2 1/4	7 1/8	8.00		372,000	62,000	11
2	6 1/4	6.30		280,000	47,700	9 3/4
1 3/4	5 1/2	4.85		224,000	37,300	8 1/2
1 5/8	5	4.15		188,000	31,300	7 3/4
1 1/2	4 3/4	3.55		164,000	27,300	7
1 3/8	4 1/4	3.00		144,000	24,000	6 3/4
1 1/4	4	2.45		116,000	19,300	6
1 1/8	3 1/2	2.00		94,000	15,700	5
1	3	1.58		76,000	12,700	4 1/2
7/8	2 3/4	1.20		58,000	9,700	4
3/4	2 1/4	0.89		46,000	7,700	3 1/4
5/8	2	0.62		31,000	5,170	2 3/4
9/16	1 3/4	0.50		24,600	4,100	2 1/3
1/2	1 1/2	0.39		20,000	3,300	2
7/16	1 1/4	0.30		16,000	2,700	1 3/4
3/8	1 1/8	0.22		11,500	1,900	1 1/2
5/16	1	0.15		7,600	1,270	1 1/3
1/4	3/4	0.10		5,300	890	1

TABLE III.

CAST STEEL FLAT HOISTING ROPE. ULTIMATE STRENGTH, WORKING STRESS AND WEIGHT OF FLAT WIRE ROPE COMPOSED OF 4 STRANDS, 7 WIRES TO THE STRAND.

Width and Thickness, In.	Weight in Lb. per Lineal Foot.	Safe Working Load for Hoisting, L, Lb.	Approximate Breaking Stress, Lb.	Safe Working Stress for Direct Pull, S, Lb.	Approximate Diameter in Inches of Round Cast Steel Rope of Equal Strength.
3/8 × 5 1/2	3.90	Safe working load, L, = $2S$ − bending stress.	110,000	18,300	1 5/16
3/8 × 5	3.40		100,000	16,700	1 1/4
3/8 × 4 1/2	3.12		94,000	15,700	1 3/16
3/8 × 4	2.86		86,000	14,300	1 1/8
3/8 × 3 1/2	2.50		76,000	12,700	1
3/8 × 3	2.00		60,000	10,000	15/16
3/8 × 2 1/2	1.86		56,000	9,300	7/8
3/8 × 2	1.19		36,000	6,000	3/4
1/2 × 7	5.90		178,000	29,700	1 5/8
1/2 × 6	5.10		154,000	25,700	1 9/16
1/2 × 5 1/2	4.82		144,000	24,000	1 1/2
1/2 × 5	4.27		128,000	21,300	1 3/8
1/2 × 4 1/2	4.00		120,000	20,000	1 5/16
1/2 × 4	3.30		100,000	16,700	1 1/4
1/2 × 3 1/2	2.97		90,000	15,000	1 1/8
1/2 × 3	2.38		72,000	12,000	1

if the cable is slack. If a descending cage should stick and then drop, the stress will be equal to the kinetic energy developed and will be very large. The load due to starting a cage suddenly from the bottom of a shaft may be taken as

$$K = 2W + R + F \tag{1}$$

where K = stress in lb. at the sheave at the instant of picking up the load;
W = gross load in lb.;
R = weight of rope in lb.;
F = friction in lb., = $(W + R)f$, where f = coefficient of friction, which may be taken at 0.01 to 0.02 for vertical shafts and from 0.02 to 0.04 for inclined shafts with the rope supported on rollers. The working load should not be greater than K plus the stress due to bending, and should not exceed $\frac{1}{3}$ of the ultimate strength of the rope, or $\frac{1}{6}$ of the ultimate strength for direct pull.

For inclined shafts with angle of inclination with horizontal = θ, the stress in the rope due to starting the cage is

$$K' = (2W + R)\sin\theta + f(W + R)\cos\theta \tag{2}$$

Bending Stresses in Wire Rope.—The stresses due to bending will depend upon the diameter of the rope, the make-up of the rope, the angle through which the rope is bent, and the diameter of the sheave. The unit stress due to bending in a round hoisting rope may be obtained from formula (3), the form of which is due to Rankine ("Machinery and Mill Work," p. 533).

$$S = 1{,}894{,}000\,\frac{d}{D} \tag{3}$$

where D = the diameter of the sheaves in inches, and d = the diameter of the rope in inches. The area of the steel in a round hoisting rope is approximately $a = 0.4d^2$, and the total bending stress in a round rope will be

$$S_b = S\cdot a = 757{,}600\,\frac{d^3}{D} \tag{4}$$

Now the direct breaking strength of a crucible steel round rope is closely

$$U = 60{,}000d^2 \tag{5}$$

Where bending stress is considered, the safe working load should not exceed $\frac{1}{3}$ of the ultimate strength, and the safe working load, L, should not exceed

$$L = 20{,}000d^2 - 757{,}600\,\frac{d^3}{D} \tag{6}$$

The safe working loads for crucible steel round ropes based on formula (6) are given in Fig. 7.*

For plough steel ropes the ultimate strength is $U = 70{,}000d^2$, and

$$L = 26{,}700d^2 - 757{,}600\,\frac{d^3}{D} \tag{6'}$$

Mr. William Hewitt in "Wire Rope," published by the Trenton Iron Company, gives the following formula for bending.†

$$S_b = \frac{E\cdot a}{1.03\,\frac{D}{d'} + C} \tag{7}$$

where E = the modulus of elasticity of steel, a = the area of the rope in sq. in., D = the diameter of the sheave in inches, d' = the diameter of the individual wires in inches, and C = a constant

* Redrawn from a diagram prepared by Mr. E. T. Sederholm, Chief Engineer, Allis-Chalmers Company.

† Also see Engineering News, May 7, 1896.

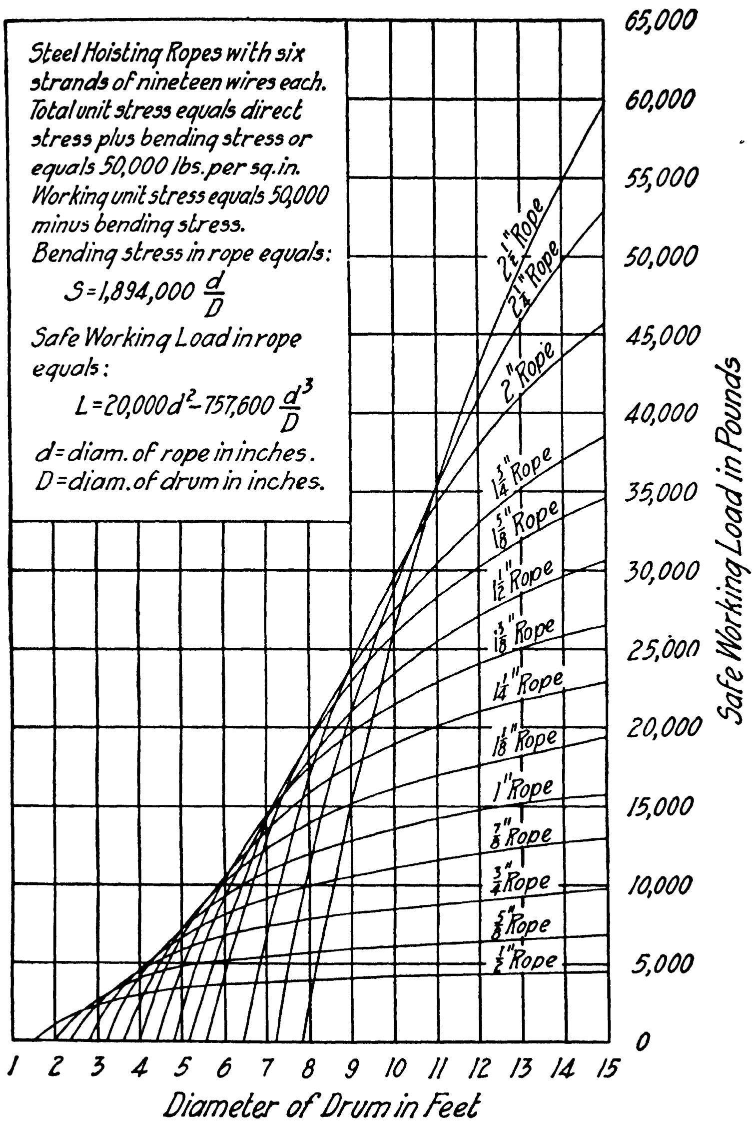


FIG. 7. SAFE WORKING STRESSES, L, IN CRUCIBLE STEEL, ROUND HOISTING ROPE.

depending upon the rope, and varies from 9.27 for haulage rope to 27.81 for tiller rope. For standard hoisting rope, $C = 15.45$. Substituting $E = 29{,}000{,}000$,

$a = 0.4\,d^2$, and $d' = \frac{d}{15}$, we have

$$S_b = \frac{750{,}000 d^3}{D - d} \tag{8}$$

Since d is very small as compared with the values of D used in hoisting, formulas (4) and (8) give practically the same results.

The bending stresses in crucible steel flat ropes are given in Fig. 8.

Cages and Skips.—For details of cages and skips, see the author's "The Design of Mine Structures."

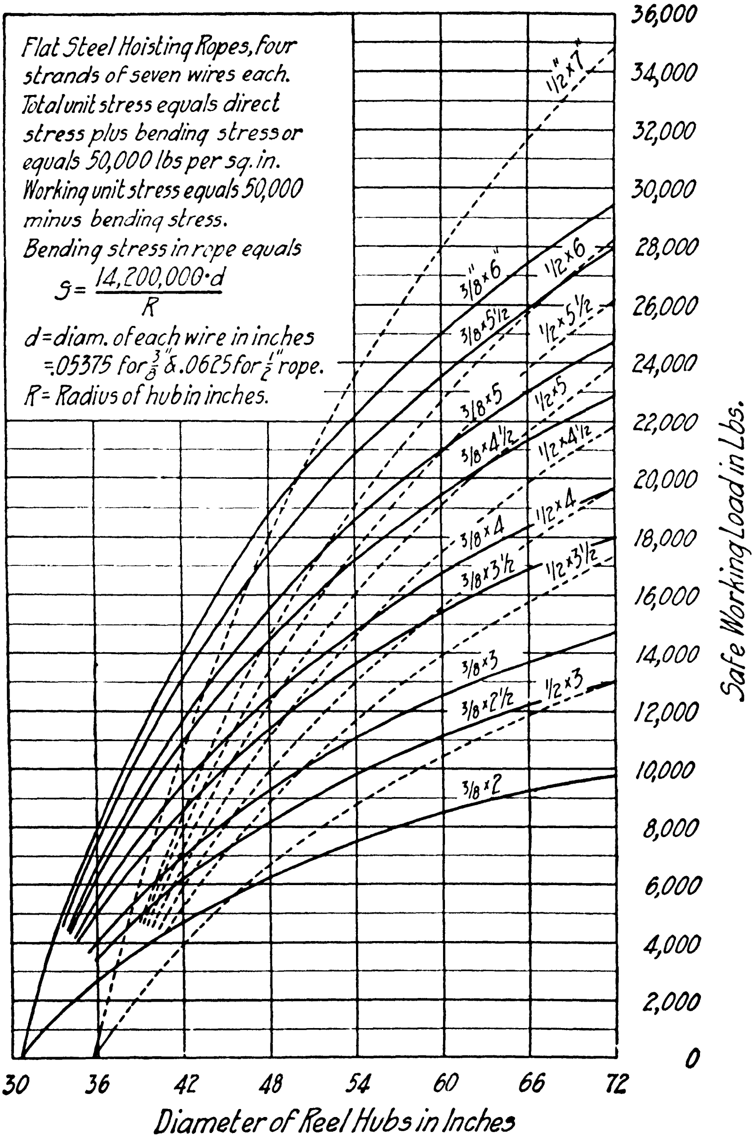


FIG. 8. SAFE WORKING STRESSES, L, IN CRUCIBLE STEEL, FLAT HOISTING ROPE.

Sheaves and Safety Hooks.—For details and data on sheaves, safety hooks, etc., see the author's "The Design of Mine Structures."

EXAMPLES OF STEEL HEAD FRAMES.—The detail plans for three steel head frames taken from the author's "The Design of Mine Structures" are excellent examples of steel head frames. Data on 16 steel head frames are given in Table V.

Steel Head Frame for the Diamond Mine.—The details of the steel head frame of the Diamond mine are shown in Fig. 9. The Diamond head frame is 100 ft. high from the collar of the shaft to the center of the sheaves. The shaft is 2,800 ft. deep. The sheaves are 10 ft. in diameter and carry a 7 in. × $\frac{1}{2}$ in. flat rope. The ore is hoisted in self-dumping skips with a capacity of 7 tons and weighing $3\frac{1}{2}$ tons, and is dumped into hoppers from which it is run directly into cars which pass beneath the head frame. The main front columns and back braces are

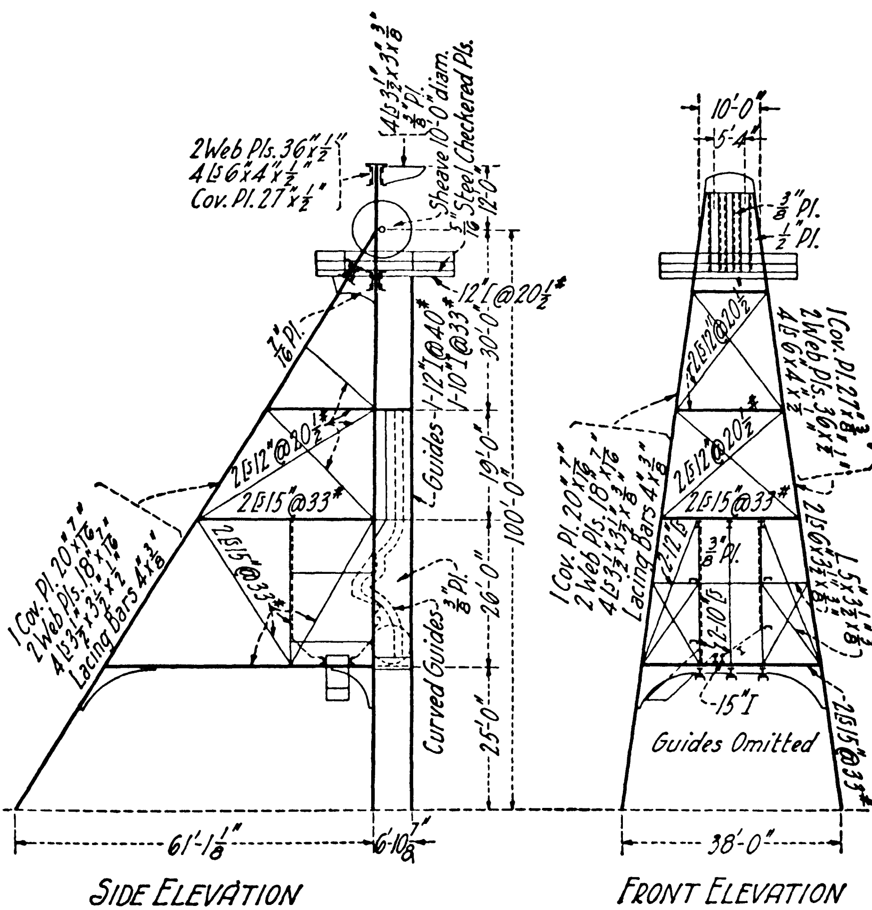


FIG. 9. STEEL HEAD FRAME FOR DIAMOND MINE, BUILT BY THE GILLETTE-HERZOG MFG. CO.

made of built-up sections consisting of one cover plate 20 in. × $\frac{7}{16}$ in., two plates 18 in. × $\frac{7}{16}$ in., angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{1}{2}$ in., with lacing bars on the inner side 4 in. × $\frac{3}{8}$ in. The main diagonal bracing is made of two channels laced. The total weight of the structural steel in the head frame proper was 292,000 lb., while the steel work in the bins weighed 26,000 lb. At 40 cts. per hour the cost of shop labor on the structural steel was 1.09 cts. per lb. The cost of erection, everything being riveted, was $11.20 per ton.

Steel Head Frame for the New Leonard Mine.—The steel head frame shown in Fig. 10 was built by the American Bridge Company for the New Leonard mine of the Boston & Montana Copper Company, Butte, Montana. The head frame is of the A-type, and is 140 ft. high from

the collar of the shaft to the center of the sheaves. The mine has a four compartment shaft, two of the compartments being used for hoisting ore. The mine is now 1,697 ft. deep, but the head frame was designed for an ultimate depth of 3,500 ft. The ore is hoisted in five-ton self-dumping skips with a single deck cage above the skip. The skips weigh 7,500 lb. each. Four-deck cages are used for hoisting men. The hoisting rope is 1½ in. in diameter, a round hoisting rope being an innovation in the Butte district. The rate of hoisting is 2,800 ft. per minute. The skip ore bins have a capacity of 150 tons. From the skip ore bins the ore runs into railroad ore bins (not shown in Fig. 14), 26 ft. 9 in. wide by 150 ft. long, with a capacity of 1,500 tons. The sheaves are 12 ft. in diameter, and are placed 5 ft. 10 in., center to center.

The main posts are made of two channels 12 in. @ 20½ lb., with a cover plate 16 in. wide and $\frac{5}{16}$ in. and ½ in. thick, with lacing on the inner side. The back braces for the lower two panels are made of channels 12 in. @ 30 lb., with a plate 16 in. × ⅜ in.; the third section is made of two channels 12 in. @ 30 lb., with a plate 16 in. × $\frac{5}{16}$ in., while the two upper sections are made of channels 12 in. @ 20½ lb., laced on both sides. The main struts and diagonal braces are made of two channels, with battens top and bottom. The skip guides are made of two channels 12 in. @ 20½ lb. The main girder at the top of the back brace consists of one plate 36 in. × ⅜ in., and four angles 4 in. × 4 in. × ⅜ in. The skip bins are supported on columns made of two channels 10 in. @ 15 lb., laced on both sides. Where two channels are used for a section, the flanges are turned out. The New Leonard head frame is one of the highest in the country, and is one of the best designed frames that has been constructed. The shipping weight of the structural steel in this head frame was 346,425 lb.

Tonopah-Belmont Steel Head Frame.—The Belmont shaft of the Tonopah-Belmont Mining Co., Tonopah, Nevada, is at present 1,420 ft. deep. It has three compartments, one for the ladder-way and pipes and two for hoisting. Double-deck cages of the Leadville type are used for hoisting, but the use of skips is contemplated later. The head frame, Fig. 11, is of the A-type, and the height is 75 ft. from the base to the center of the sheaves. The hoisting drum is placed 100 ft. from the center of the shaft.

TABLE IV.

ESTIMATE OF WEIGHT OF 75-FT. STEEL HEAD FRAME, TONOPAH-BELMONT MINING CO.

Member.	Weight in Lb.		Total Weight, Lb.	Details in Per Cent of Main Members.
	Main Members.	Details.		
Back braces	9,170	4,150	13,320	43
Front posts	3,590	2,790	6,380	77
Girders	5,446	1,250	6,696	23
Diaphragms	2,936	2,582	5,518	82
Channels	1,790	440	2,230	25
Angle struts	2,627	1,015	3,642	39
Channel struts	3,263	2,179	5,442	67
Stringers	1,466	613	2,079	43
Angle bracing	8,065	2,279	10,344	28
Steel girders	6,673	414	7,087	6
Total	45,026	17,712	62,738	39.4

The sheave wheels are of the bicycle pattern with a diameter of 84 in. at the center of the rope groove, and an over all diameter of 91 in. Each wheel has 16 spokes of 1¼ in. rolled iron rods. The spokes are cast at their inner ends into two rings 16 in. in diameter and 3 in. wide, so that they form integral parts of the hub, which is 12 in. in diameter and 16 in. long, while the outer ends are cast into bosses on the inside of the ring. The rolled steel shafts are 16 in. in diameter at the central portion with bearings 5 in. in diameter, and are 12 in. long. The rope grooves are turned in hard maple blocks fastened in a recess in the rim. The total weight of the sheaves is 2,950 lb. each.

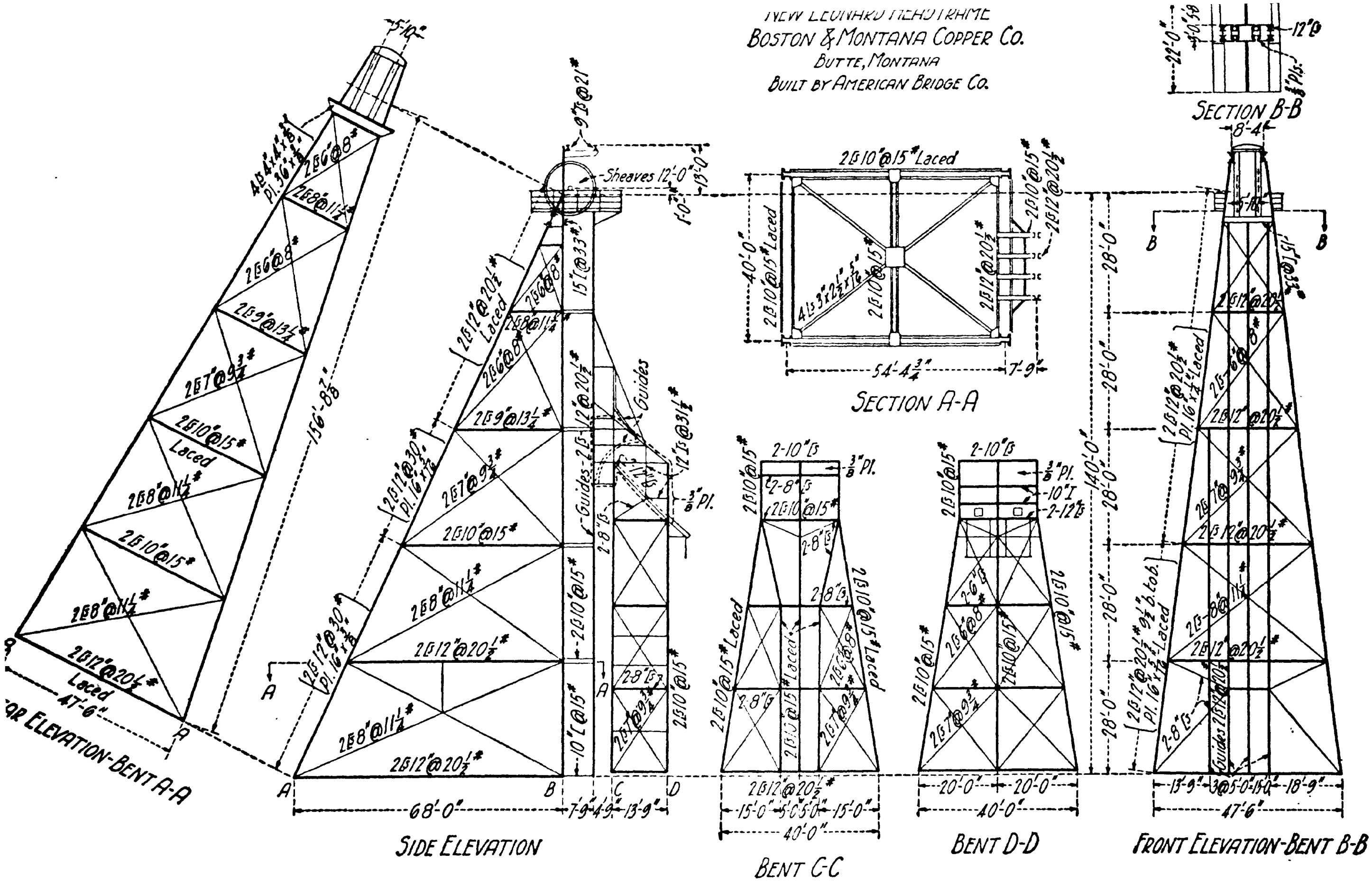


FIG. 10. NEW LEONARD STEEL HEAD FRAME.

The head frame is designed so as to give a factor of safety of 8 when there is on each sheave a load of 100,000 lb. The head frame is sufficiently strong and rigid to permit of hoisting loads of 7 tons from a depth of 2,000 ft. at a speed of 1,000 ft. per minute without appreciable vibration during the most severe period of starting and acceleration.

TABLE V.

DATA ON STEEL HEAD FRAMES.

	Description.	Depth of Mine, Ft.	Height of Frame, Ft. In.	Diameter of Sheaves, Ft. In.	Size of Hoisting Rope, In.	Method of Hoisting.	Weight of		Weight of Ore, Lb.	Rate of Hoisting.		Weight of Head Frame, Lb.
							Skip, Lb.	Cage, Lb.		Ft. per Min.	Tons per Day.	
1	Sibley Mine, Ely, Minn...	726 (designed for 2,000)	140-0	12-0	$1\frac{3}{8}$	Skips	5,000	3,500	14,000	2,000	....	576,663
2	High Ore, Butte, Mont...	2,800	100-0	10-0	$7\times\frac{1}{2}$	Skips	7,000	....	14,000	1,000	1,200	292,000
3	Diamond, Butte, Mont...	2,800	100-0	10-0	$7\times\frac{1}{2}$	Skips	7,000	....	14,000	1,000	1,200	318,000
4	New Leonard, Butte, Mont.	1,679 (designed for 3,500)	140-0	12-0	$1\frac{1}{2}$	Skips	7,500	....	10,000	2,800	....	346,425
5	Inland Steel Co., Hibbing, Minn.	225	76-0	6-0	$1\frac{1}{8}$	Skips	3,700	....	6,700	....	....	79,000
6	Elkton, Elkton, Colo.....	.....	55-0	5-0	$3\frac{1}{2}\times\frac{3}{8}$	.....	.....	....	15,200 working load	....	...	......
7	Cia. Minera de Penoles, Bermejillo, Mex.......	1,000	90-0	7-0	$1\frac{1}{8}$	Skips	5,000	....	10,000	....	....	80,000
8	Tonopah-Belmont, Tonopah, Nev.	1,420	75-0	7-0	1	.....	.....	....	.....	1,000	....	63,000
9	Copper Queen, Bisbee, Ariz.	1,700	60-0	7-0	$1\frac{1}{4}$	Skips	5,990	....	3,700	....	2,000	35,250
10	Union Shaft, Virginia, Nev.	[illegible],000	50-0	7-0	1	Cages	.....	1,200	2,400	1,000	500	42,000
11	Speculator, Butte, Mont..	.....	50-0	7-0	$7\times\frac{1}{2}$	Skips	.....	....	.....	.....	....	42,200
12	Basin & Bay State, Basin, Mont.	.....	70-0	10-0	$1\frac{1}{4}$	Skips	.....	....	.....	.....	....	79,000
13	Steward, Butte, Mont....	.....	55-0	7-0	.....	Skips	...	....	10,000	.....	....	45,000
14	Anaconda, Butte, Mont. .	2,400	58-8	10-0	$7\times\frac{1}{2}$	Skips	7,000	....	14,000	1,000	1,200	74,700
15	Quincy Rock House, No. 2, Hancock, Mich......	6,000 (inclined 57°)	119-3	12-0	$1\frac{1}{2}$	Skips	10,000	....	168 cu. ft.	....	2,400	839,000
16	St. Lawrence, Butte, Mont.	2,100	97-0	10-0	$7\times\frac{1}{2}$	Skips	7,000	....	14,000	1,000	1,200	117,000

The head frame was built by the Koken Iron Works, St. Louis, Mo., was made of structural steel furnished under standard specifications, and was fully riveted up in place with pneumatic hammers. The shipping weight of the structural steel was 63,000 lb.

The hoist is placed 100 ft. from the shaft, and is a Wellman-Seaver-Morgan double drum electric hoist with drums having 64 in. diameter and a face 36 in. wide between flanges. The hoist is designed to operate in or out of balance and is capable of handling a load of 12,000 lb. at a speed of 1,000 ft. per minute. The hoisting rope is a six strand, nineteen wire, plow-steel rope, 1 in. in diameter, that weighs 1.58 lb. per ft., and each rope is 1,700 ft. long. The diameter

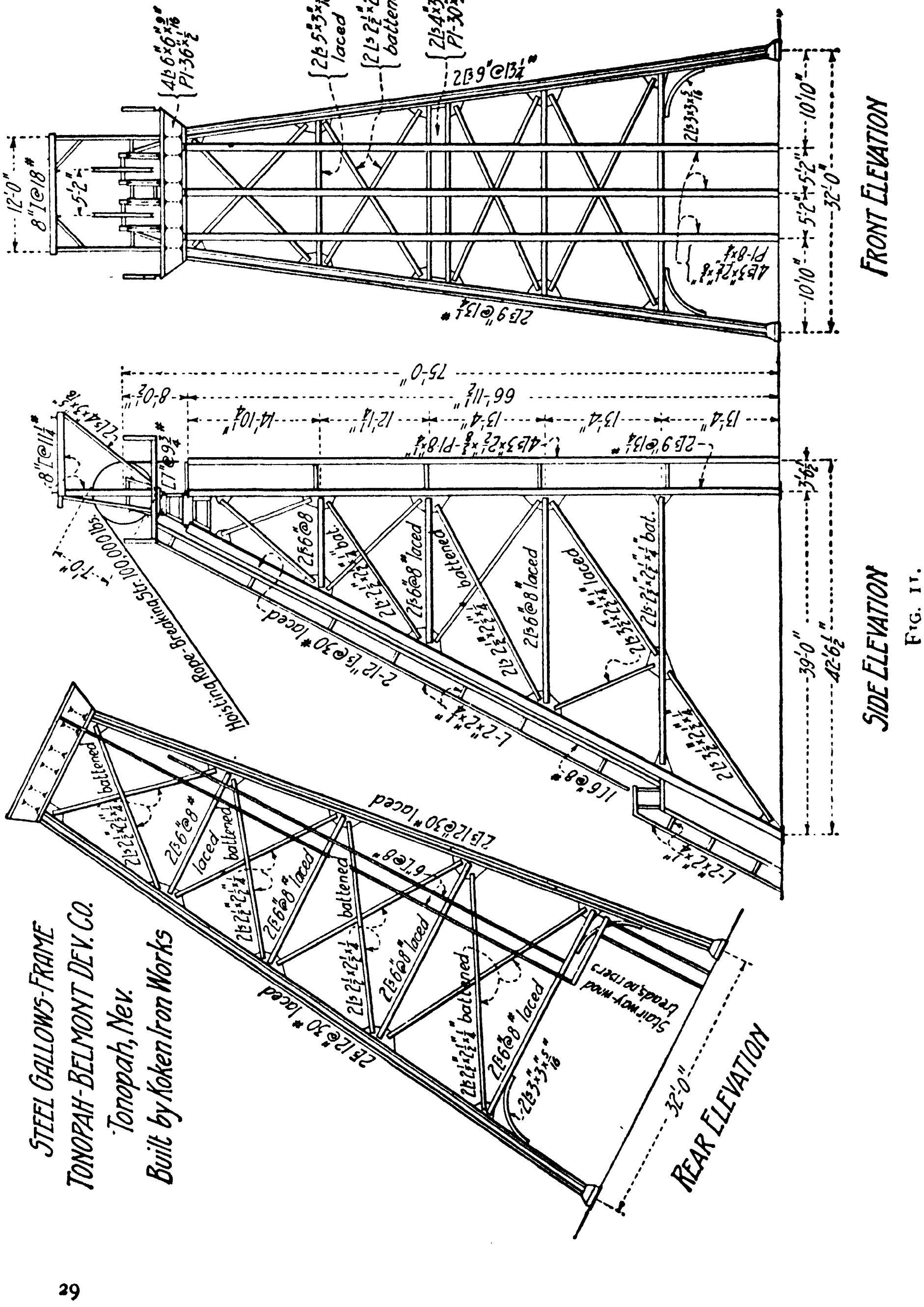

FRONT ELEVATION
SIDE ELEVATION
REAR ELEVATION
STEEL GALLOWS-FRAME
TONOPAH-BELMONT DEV. CO.
Tonopah, Nev.
Built by Koken Iron Works
Hoisting Rope-Breaking Str. 100,000 lbs.
Stairway-wood treads on risers

FIG. 11.

of the drum at the hoist is 64 in., but the rope winds twice around the drum, so that the diameter is 66 in. near the end of the lift. With proper allowance for bending stresses the working stresses under the most severe conditions do not exceed the working load of 7.6 tons as given by the manufacturers of the wire rope.

Estimate of Weight of a Steel Head Frame.—A summary of a detailed estimate of the 75 ft. steel head frame built by the American Bridge Company at Tonopah, Nev., is given in Table IV. The details are 39.4 per cent of the weight of the main members. The rivet heads are 4.1 per cent of the weight of the structure.

For additional examples of steel head frames, see the author's "The Design of Mine Structures."

COAL TIPPLES.—The design of a coal tipple depends upon the quality of the coal, upon whether the coal is hoisted from the shaft or is taken from a drift or tunnel, and upon the work that it is necessary to do in order to prepare the coal for the market. The coal tipple for a bituminous mine in which the coal is hoisted from a shaft, consists of a head frame and a shaker structure or tipple proper where the coal is weighed and screened. A coal tipple for an anthracite mine ordinarily consists of a head frame with storage bins into which the coal is run without crushing or screening; the coal being prepared for market in a separate breaker building. Where bituminous coal is dirty or contains a large amount of refuse material it is sometimes cleaned in a washer building, or is broken, sized and cleaned in a coal breaker.

With a double compartment shaft the shaking structure, or tipple proper, is usually placed with its axis at right angles to the center line of the two compartments. The hoisting ropes may be either parallel to the axis of the tipple, in which case the head sheaves are parallel; or may be placed at right angles to the axis of the tipple, in which case the sheaves are placed in tandem. The coal may be run through rotary screens, or over shaking screens as is now the common practice. Shaking screens are usually divided into sections and are driven by eccentrics placed 180 degrees apart. The shaking screens do not ordinarily weigh more than two to three tons empty or four to six tons when loaded, but are driven with a velocity of 100 to 150 strokes per minute, with a length of stroke of from 4 to 12 in. and the shaking motion makes it necessary to design the shaker structure with great care in order to reduce the vibration. The best modern practice in the design of coal tipples is to make the head frame and the tipple, or shaker structure, entirely separate and independent units.

Sizing Coal.—The object in sizing coal is to separate the dirt and slack from the coal, and to obtain a product that can be burned more advantageously than unsized coal. A compact coal will not admit the air and will burn on the surface, and it is therefore an advantage to have the lumps of approximately equal size. The sizes and names of the different grades of coal differ considerably in different localities.

Types of Coal Tipples.—Coal tipples may be classed under three types, depending upon the manner in which the coal is brought to the tipple; (1) hoisting in cages or skips from vertical or slightly inclined shafts; (2) cage hoisting on an incline either from a shaft, or on a bridge, or from a tunnel; (3) conveyor hoisting either from the mine or from a head bin into which the coal has been dumped from cars or skips.

The design and operation of coal tipples will be illustrated by describing three steel coal tipples. (1) Steel Coal Tipple for the W. P. Rend Coal Company—vertical hoisting with self dumping cages and shaking screens; (2) Spring Valley No. 5 Steel Coal Tipple—vertical hoisting in cages, with Ramsey transfer and shaking screens; and (3) Phillip's Coal Tipple—vertical hoisting with self dumping cages dumping into a storage bin.

Steel Coal Tipple for W. P. Rend Coal Company.—The steel coal tipple for the W. P. Rend Coal Company, Rendville, Ill., has the head frame covering four tracks, with provision for four extra tracks on the opposite side of the center line of the head frame. The steel head frame is 79 ft. 6 in. from the collar of the shaft to the center of the sheaves. The sheaves are 8 ft. in diameter and carry a 1¾ in. hoisting cable.

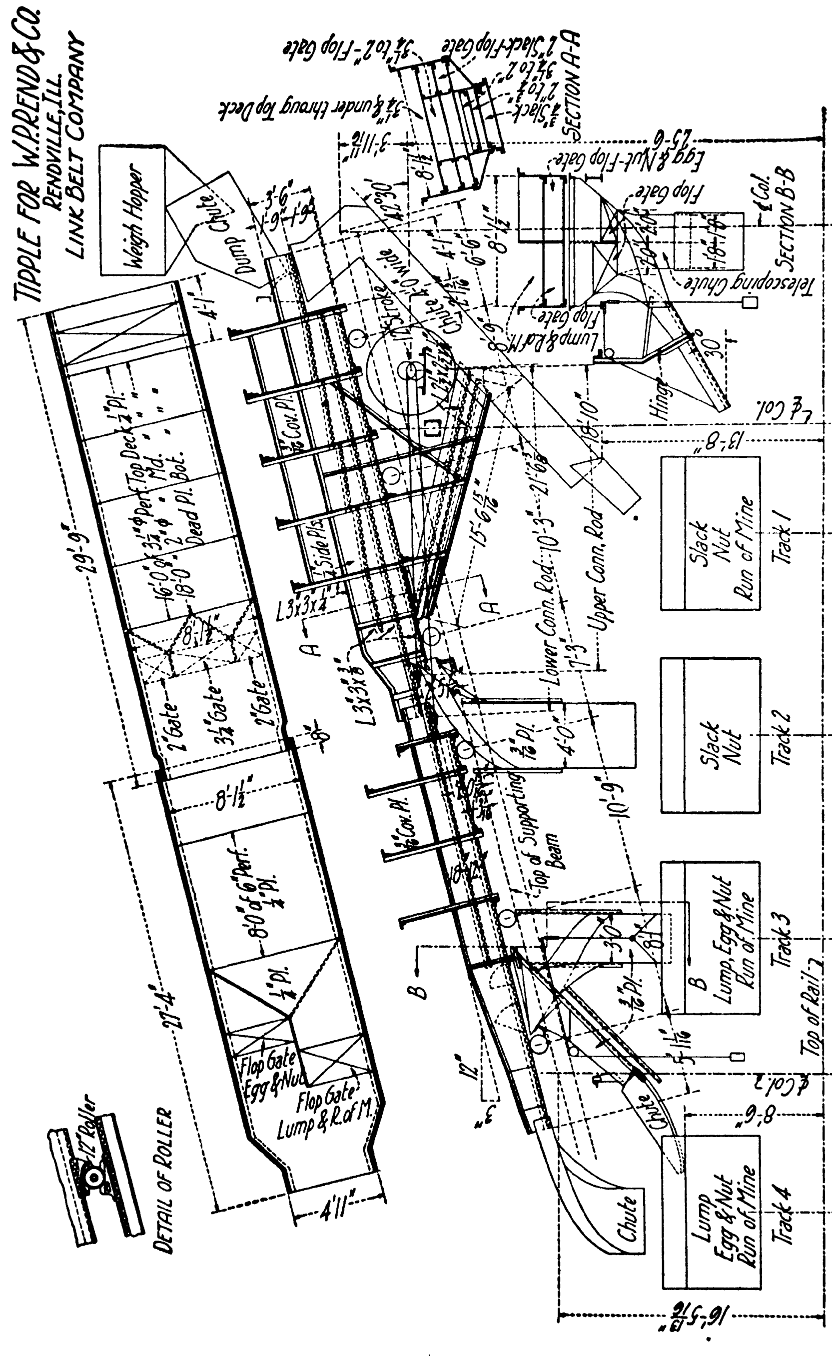


FIG. 12 SHAKING EQUIPMENT FOR W. P. REND COAL TIPPLE.

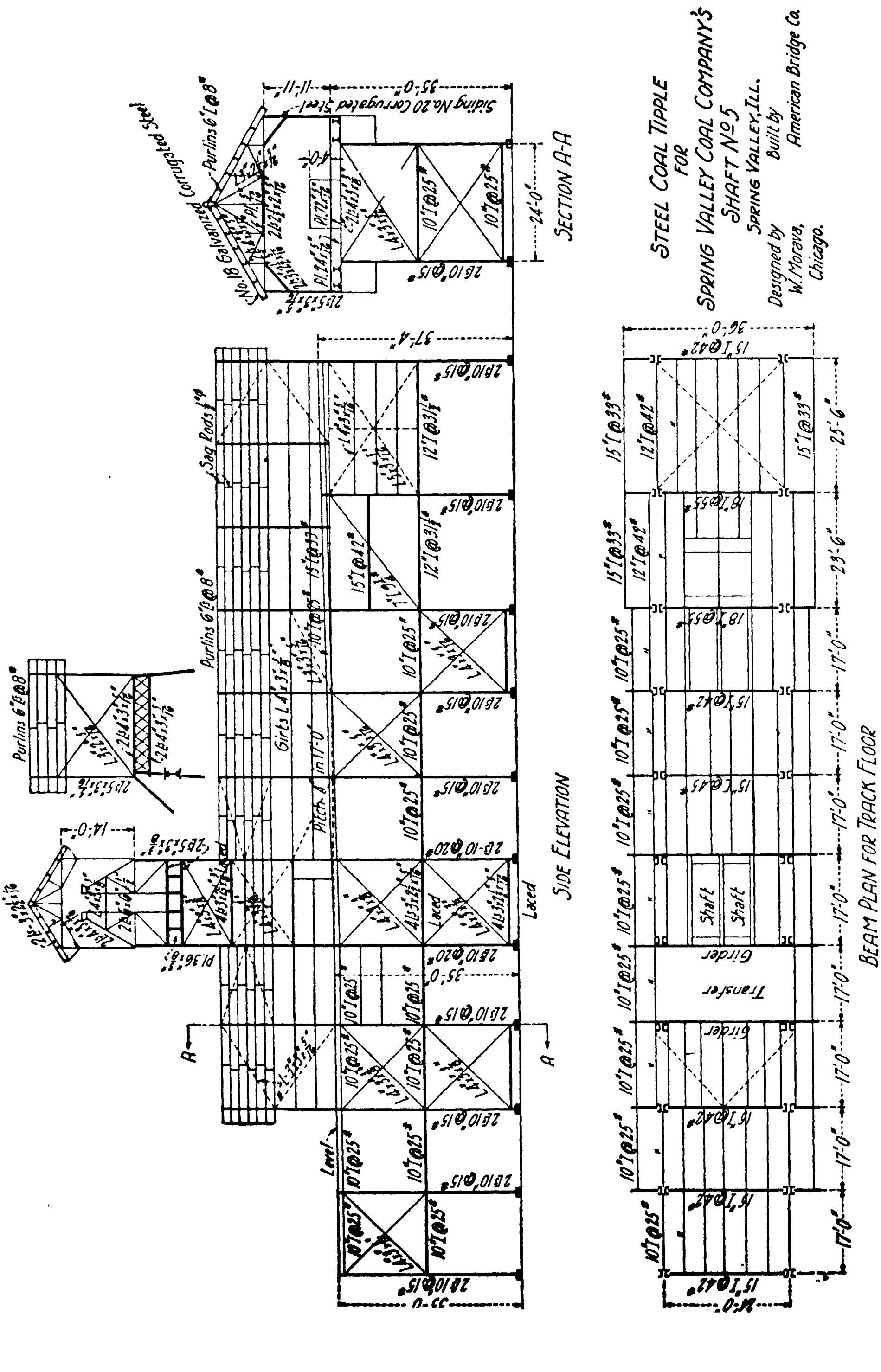


FIG. 13. STEEL FRAMEWORK FOR SPRING VALLEY NO. 5, COAL TIPPLE.

Operation of Coal Tipple.—Detail plans of the shaking screens and tipple equipment are shown in Fig. 12. The coal is raised from the mine in self dumping cages and is dumped into two weigh hoppers having a capacity of four tons each. From the weigh hoppers the coal passes through a dump chute, and may be run directly into cars on the track or may be run over shaking screens. The first section of the shaking screens is 29 ft. 9 in. long, the top deck, having a length of 16 ft., has ¾ in. round perforations; the middle, having a length of 18 ft., has 2 in. round perforations, the bottom plate being solid. The upper deck of screens sloping toward the head frame has perforations 3½ in. to 2 in. round; the second deck has perforations 2½ in. to 3 in. round; the third plate deck has perforations ¾ in. round, the bottom deck being solid. The coal passing over the 2 in. and 3½ in. round perforations of the main screen may be run back over the shaking screens just described, or may be run over the second shaking screen 27 ft. 4 in. long and 8 ft. wide. This shaking screen has a length of 8 ft. with perforations 6 in. in diameter. By making different combinations of the screens different grades of coal can be obtained, as is shown in Fig. 12. The shaking screens are carried on rollers 12 in. in diameter, which are operated by eccentric connecting rods with a 12 in. stroke. These rollers give the shaking screens a motion in two directions and give much more satisfactory results than the earlier method of suspending the shaking screens from overhead supports. The capacity of the tipple is 2,500 tons in eight hours.

The tipple was designed and constructed by the Wisconsin Bridge & Iron Company, and the tipple equipment was furnished by the Link-Belt Company.

Steel Coal Tipple at Spring Valley Shaft No. 5.—The steel coal tipple constructed at Spring Valley shaft No. 5, Spring Valley, Illinois, is one of the best examples of steel tipple construction for bituminous mines. The steel tipple building is 187 ft. long, 36 ft. wide and 35 ft. from the track level to the level part of the main tipple floor. The steel head frame is 75 ft. and 85 ft. 6 in. from the track level to the centers of the sheaves, respectively. The sheaves are 10 ft. in

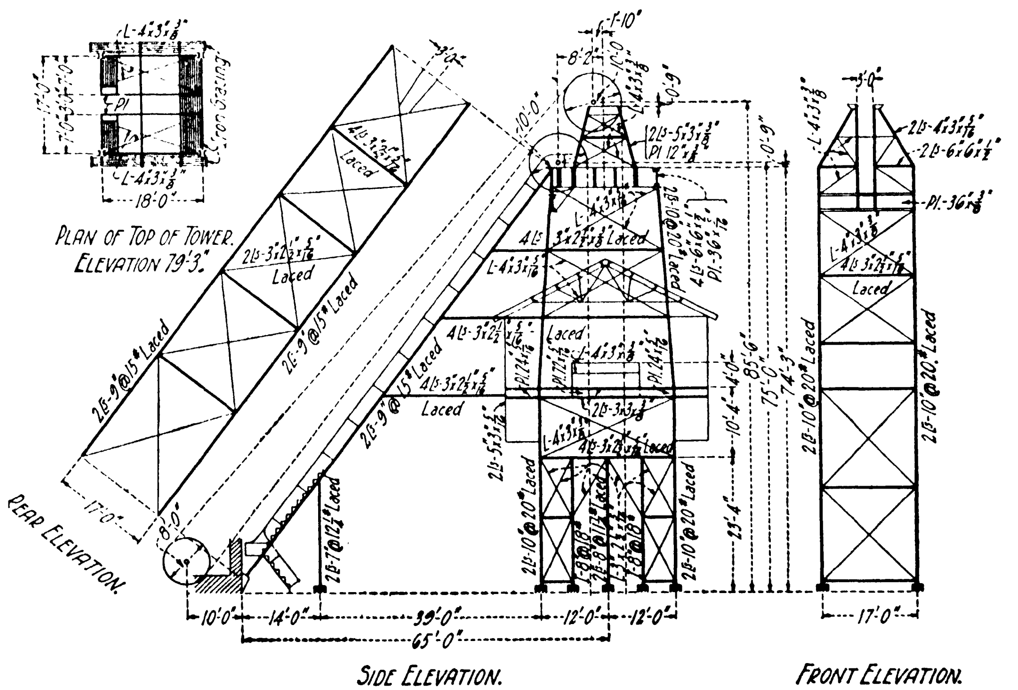


FIG. 14. STEEL HEAD FRAME, SPRING VALLEY COAL TIPPLE, SHAFT NO. 5.

diameter and are placed tandem with the hoisting rope, and at right angles to the axis of the main tipple building. The hoisting rope is crucible steel 1⅜ in. in diameter. The steel tipple building and head frame are covered with No. 18 galvanized corrugated steel carried on steel purlins. Detail plans of the tipple structure are given in Fig. 13 and of the head frame in Fig. 14. The head frame and tipple building are fully braced and make a very rigid structure. The main track floor of the tipple is level over the first five panels on the left of the structure, the remainder of the floor having a pitch of 4 in. in 17 ft. The tipple floor is covered with 4 in. planking spiked to 4 in. nailing strips which are carried on I-beam joists. The weight of the structural steel, including the corrugated steel but not including tipple equipment, was 415,530 lb.

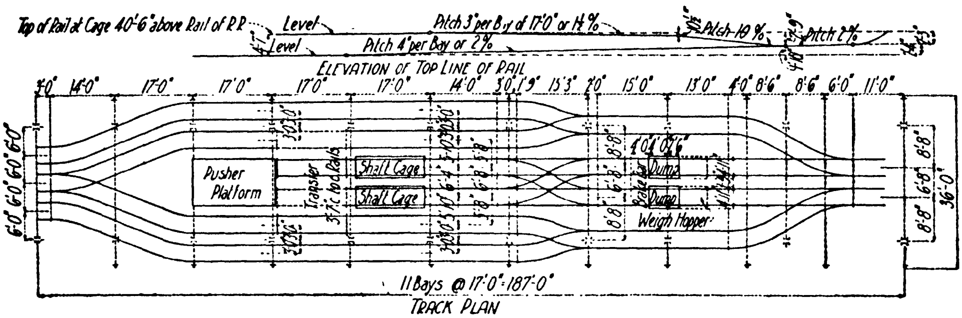


FIG. 15. PLAN OF TIPPLE TRACKS, SPRING VALLEY NO. 5 COAL TIPPLE.

Operation of Tipple.—The detail track plan is shown in Fig. 15; the operation of the Ramsey transfer is shown in Fig. 16, and the arrangement of the shaking bar screens is shown in Fig. 17. Two coal cars containing 1½ tons each are hoisted on the shaft cage. The loaded cars are pushed off the cage and two empty cars are pushed on the cage by means of a steam pusher, as shown in Fig. 16. From the cage platform the loaded cars run by gravity on a 1½ per cent grade to the dumps, where the coal is dumped by Phillips automatic tipples or dumps. After dumping, the cars pass to the right by gravity on the 10 per cent descending grade and are stopped by a 2 per cent ascending grade and a short piece of track. The cars then return by gravity, and may either be switched to the outside tracks or run back on the transfer tracks. The empty cars are run on the platform of the Ramsey transfer and are raised by a steam cylinder a height of 4 ft. 7 in. to the level of the floor of the shaft cage, and are ready to be shoved on the cage by the steam pusher.

The coal is dumped by the Phillips tipple dumps into one of two weigh hoppers 5 ft. wide, as shown in Fig. 17. After the coal is weighed it runs out of the weigh hopper on a converging chute having a slope of 30 degrees with the horizontal. From the converging chute the coal runs over shaking bar screens 6 ft. 6 in. wide, the bars being placed ⅞ in. apart. The fine coal passing through this screen runs over a ⅜ in. shaking bar screen and is chuted into the cars. The slack passing through the ⅜ in. bar screen is run directly into the cars. From the ⅞ in. shaking bar screen the lump coal passes through a converging chute and over a bar screen 5 ft. 6 in. wide with the bars spaced 5 in. apart, from which the lump coal is run into cars. It will be noted that five grades of coal are obtained: mine run coal; lump coal passing over the 5 in. screen; coal passing the 5 in. screen and retained on a ⅞ in. screen; nut coal passing a ⅞ in. screen and retained on a ⅜ in. screen, and slack.

The capacity of the coal tipple is from 1,800 to 2,000 tons per day. The tipple was designed by Mr. W. Morava, Consulting Engineer, Chicago, Ill., and was built by the American Bridge Company in 1900.

Steel Coal Tipple for the Phillips Mine.—The steel coal tipple at the Phillips mine of the H. C. Frick Coke Company is an excellent example of a modern coal tipple for handling bituminous coal. Detail plans of the coal tipple are shown in Fig. 18. The steel head frame is of the 4-post

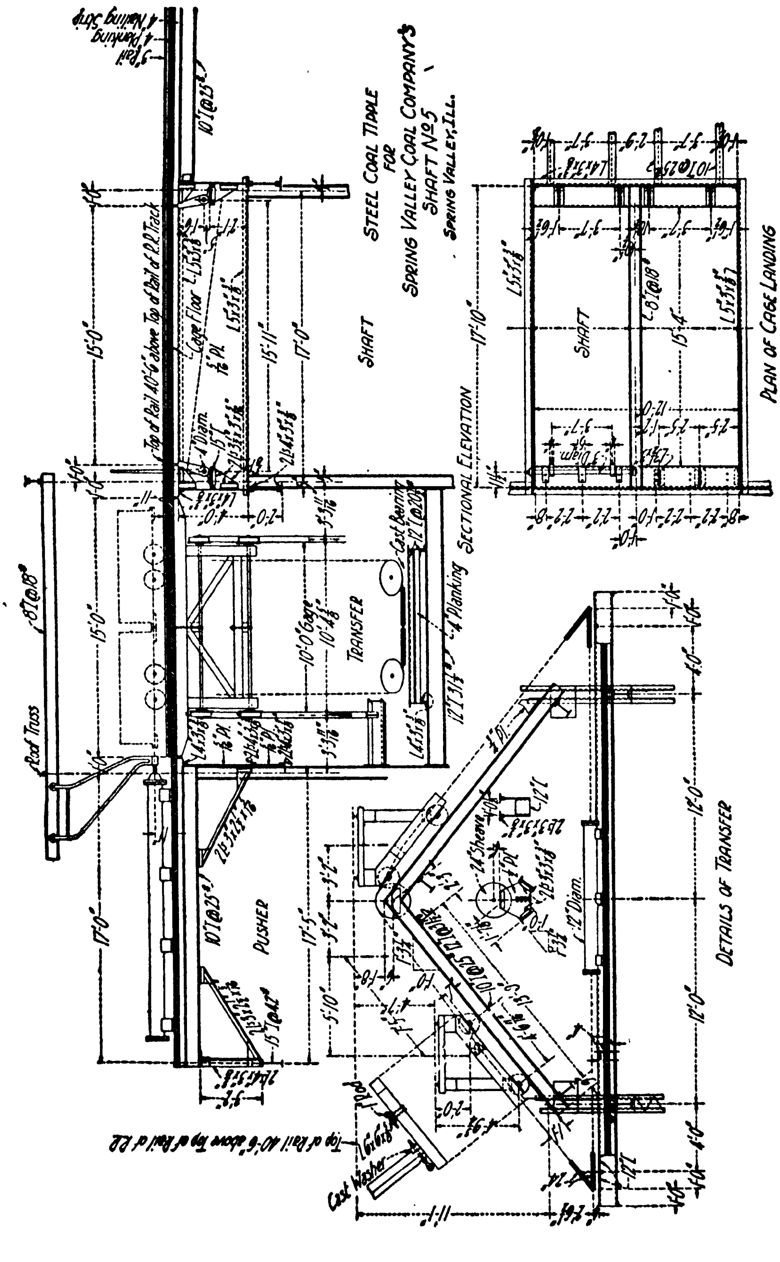


FIG. 16. RAMSEY TRANSFER, SPRING VALLEY NO. 5 COAL TIPPLE.

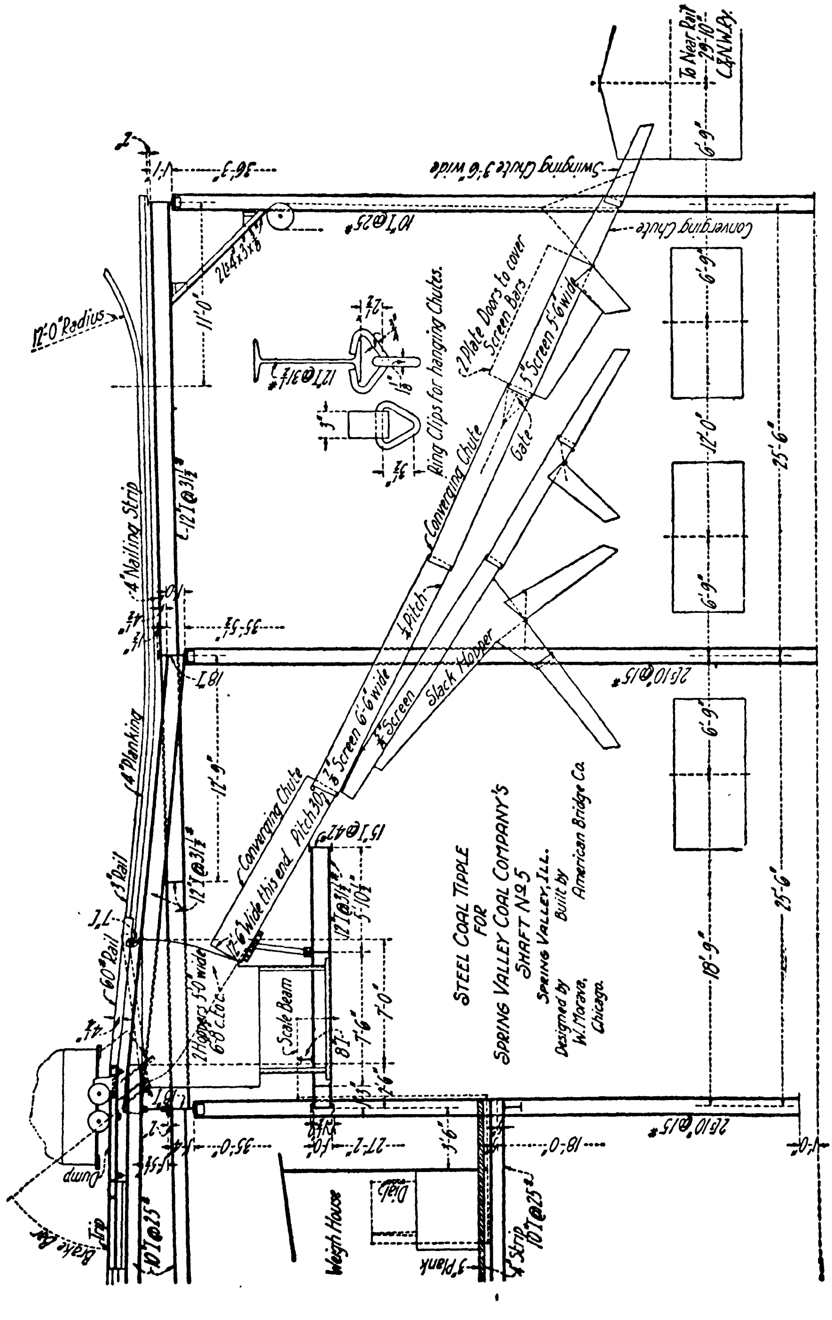


FIG. 17. SHAKING SCREENS, SPRING VALLEY NO. 5 COAL TIPPLE.

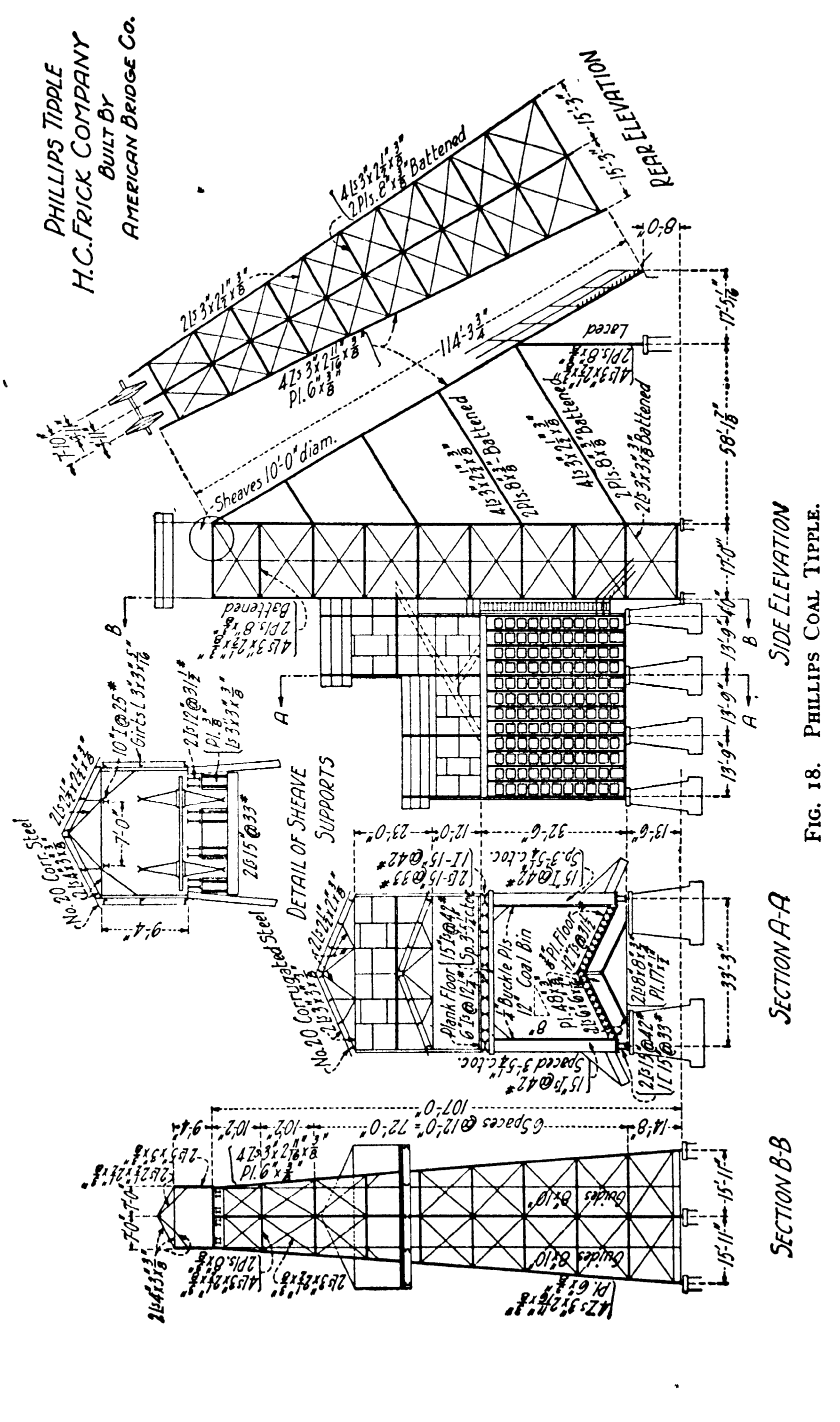


FIG. 18. PHILLIPS COAL TIPPLE.

type, and is 107 ft. from the collar of the shaft to the center of the sheaves. The main tower of the head frame has six posts made of 4 Z's 3 in. × $2\frac{11}{16}$ in. × $\frac{3}{8}$ in. with one plate 6 in. × $\frac{3}{8}$ in. The back braces consist of three columns having the same section as the main posts. The head frame is fully cross-braced with angle struts, as shown in Fig. 22. The batter of the main tower columns is 1 in. in 12 in., while the back brace makes an angle of 30 degrees with the vertical. The sheaves are 10 ft. in diameter and are supported on I-beams, resting at the end nearest the engine house on a built-up frame of angles and plates carried on two 15 in. I-beams, so as to make the necessary clearance for the sheaves. The roof trusses above the sheaves carry two I-beams, on the lower flanges of which are trolleys arranged for the attachment of chain blocks for placing and replacing the sheaves. The shipping weight of structural steel, including the corrugated steel, was 569,500 lb.

TABLE VI.

DATA ON STEEL COAL TIPPLES.

	Depth of Mine, Ft.	Height of Head Frame, Ft. In.	Diameter Sheaves, Ft. In.	Size of Hoisting Rope, In.	Method of Hoisting.	Weight of Cage Skip, Lb.	Weight of Coal, Lb.	Rate of Hoisting. Ft. per Min.	Rate of Hoisting. Tons per Day.	Weight of Structure in Lb.
Phillips Coal Tipple, Pennsylvania......	268	107–0	10–0	$1\frac{3}{8}$	Self dumping cages	......	4,000	.....	6 tons per min.	569,500
Philadelphia & Reading, Gilberton	1,100	55–0 66–0	14–0	2	......	40,000 working load each compartment		2,300	.....	......
Cardiff No. 2, Cardiff, Ill...............	.....	65–9 74–3	10–0	$1\frac{3}{8}$	Cars			.....	2,000	180,000
Spring Valley No. 5, Spring Valley, Ill...	.....	75–0 85–6	10–0	$1\frac{3}{8}$	......	......	......	.....	2,000	415,530
Alberta Railway & Irrigation Co., Lethbridge, Alta.......	600	83–0 95–0	12–0	$1\frac{1}{2}$	Cars	2,000	8,000	.....	200 tons per hour	500,000
Rend Tipple, Rendville, Ill...........	.....	79–6	8–0	$1\frac{3}{8}$	Self dumping cages	......	......	.....	2,500	Head Frame 100,000 Shaker 56,000
Carbon Tipple, Carbon, Montana.....	.....	90–0	9–0	...	Cars	......	......	.....	.....	355,400 Structural steel 16,800 Corrugated steel
R. F. C. Co. Tipple, Montana..........	.....	......	.....	...	Cars	......	......	.....	.....	171,200 Structural steel 31,300 Corrugated steel
Gebo Tipple, Montana	.....	......	.....	...	Cars	......	......	.....	.....	117,200 Structural steel 10,300 Corrugated steel

The coal is hoisted in self-dumping cages which dump the coal into distributing chutes, in which it runs by gravity to the bins having a capacity of 800 tons. The coal, being all used for making coke, is not screened or weighed.

The storage bins are built with a steel framework and are lined with $\frac{1}{4}$ in. buckle plates on the sides, and have a $\frac{3}{8}$ in. plate floor. The sides are supported by the 15 in. I-beams @ 42 lb., spaced 3 ft. $5\frac{1}{4}$ in. center to center. The inclined bottom framing consists of girders having 48 in. × $\frac{3}{8}$ in. web plates and flanges composed of two angles 6 in. × 6 in. × $\frac{7}{16}$ in., and are tied together with ties consisting of two angles 8 in. × 8 in. × $\frac{3}{4}$ in. and one plate 17 in. × $\frac{1}{2}$ in. at the bottom,

and 15 in. I-beams @ 42 lb. at the top, the girders being spaced 3 ft. 5½ in. center to center. The main side girders are composed of two I-beams 15 in. @ 42 lb., and one channel 15 in. @ 33 lb. The ⅜ in. plate floor is carried on 12 in. I-beams spaced about 1 ft. 6 in. centers. The steel plate floor is placed at a slope of 8 in. in 12 in., and it is stated that 95 per cent of the coal can be withdrawn from the bin. The bins discharge through vertical gates in the sides into motor-driven larries, which run to the coke ovens. The vertical gates are raised by rack and pinion and chain wheels.

Data on ten steel coal tipples are given in Table VI. For additional examples and data on steel coal tipples, see the author's "The Design of Mine Structures."

SPECIFICATIONS FOR STEEL HEAD FRAMES AND COAL TIPPLES, WASHERS AND BREAKERS.*

PART II.

BY

MILO S. KETCHUM,
M. Am. Soc. C. E.

1912

GENERAL DESCRIPTION.

198. **Types of Structure.**—The structure shall be of a type that will give maximum rigidity and strength. The structure shall be of a type in which the stresses can be calculated either by statics or by taking into account the deformations of the members.

199. **Bracing.**—All bracing shall be stiff, and shall be riveted together at all intersections to give maximum rigidity.

200. **Proposals.**—Contractors in submitting proposals shall furnish complete stress sheets, general plans of the proposed structures, giving sizes of material, and such detail plans as will clearly show the dimensions of the parts, modes of construction and sectional areas.

201. **Detail Plans.**—The successful contractor shall furnish all working drawings required by the engineer free of cost. Working drawings will, as far as possible, be made on standard size sheets 24 in. × 36 in. out to out, 22 in. × 34 in. inside the inner border lines.

202. **Approval of Plans.**—No work shall be commenced or materials ordered until the working drawings are approved in writing by the engineer. The contractor shall be responsible for dimensions and details on the working plans, and the approval of the detail plans by the engineer will not relieve the contractor of this responsibility.

Loads.

203. The structures shall be designed to carry the following loads without exceeding the permissible unit stresses.

204. **Dead Loads.**—The dead loads shall consist of the weight of the head sheaves, sheaves, blocks and girders, the weight of the structure, and all concentrated machinery and equipment loads.

205. **Working Loads.**—The working loads on head frames for vertical shafts shall be taken as equal to

$$K = 2W + R + (W + R)f \qquad (1)$$

where K = the working stress in lb. at the head sheave at the instant of picking up the load; W = the gross load of the cage or skip and the load of ore or coal in lb.; R = the weight of the rope from the head sheaves to the bottom of the shaft in lb.; and f = coefficient of friction of the rope, skip and sheaves, which may be taken at 0.01 to 0.02 for vertical shafts and 0.02 to 0.04 for inclined shafts with ropes supported on rollers.

206. For inclined shafts the working load shall be taken as

$$K' = (2W + R) \sin\theta + f(W + R) \cos\theta \qquad (2)$$

where θ = the angle of inclination of the shaft with the horizontal.

* From Specifications for Steel Mine Structures as printed in the author's "The Design of Mine Structures." Part I is "Specifications for Steel Frame Buildings" as printed in Chapter I.

207. **Breaking Load.**—The head frame shall be designed for a load in one or all of the hoisting ropes equal to the breaking stress of the hoisting rope as given in the manufacturer's catalog.

208. **Machinery Loads.**—The stresses due to machinery, crushers, tipple equipment, etc., shall be considered the same as the stresses due the working or live load.

209. **Wind Loads.**—Where the head frame or tipple is enclosed the wind load shall be assumed as 30 lb. per sq. ft. of exposed surface acting horizontally. Where the framework is open the wind load shall be taken as 50 lb. per sq. ft. acting on the projection of the members of the head frame or tipple. In calculating the stresses due to wind, the wind loads may be assumed as applied at the joints of the structure. Where one side of the structure is open so that a deep cup or pocket is formed the wind load shall be taken as not less than 60 lb. per sq. ft. on the projection of the cup-like surface.

210. **Snow Loads.**—Snow loads shall be taken the same as for steel frame buildings.

ALLOWABLE UNIT STRESSES.

211. Steel head frames, coal tipples, coal washers and breakers, and similar structures shall be designed for the following allowable stresses.

212. **Dead Load Stresses.**—The allowable unit stresses for dead loads shall be the same as for steel frame buildings given in "Specifications for Steel Frame Buildings." Snow loads shall be considered as dead loads.

213. **Working Load Stresses.**—The allowable unit stresses for working loads shall be one-half the allowable unit stresses for dead load stresses as given in "Specifications for Steel Frame Buildings."

214. **Bins.**—Bins shall be designed for two thirds the allowable unit stresses for dead load stresses as given in "Specifications for Steel Frame Buildings."

215. **Breaking Load Stresses.**—The allowable unit stresses for the maximum stresses due to breaking one or all the hoisting ropes shall be equal to the allowable unit stresses for dead load stresses, plus 50 per cent, equal to three times the allowable unit stresses for working loads. The breaking loads and working loads for any shaft compartment or machine need not be assumed as acting together.

216. **Machinery Load Stresses.**—The allowable unit stresses for the maximum stresses due to machinery and moving loads shall be the same as the allowable unit stresses for working loads, equal to one half the allowable unit stresses for dead load stresses.

217. **Wind Load Stresses.**—The allowable unit stresses when the wind load stress is combined with the dead load stress plus twice the working load and machinery load stresses shall not exceed the allowable unit stresses for dead loads by more than 25 per cent. If the sum of the wind load unit stress, the dead load unit stress, and twice the working load and machinery load unit stresses exceed the allowable unit stress for dead loads by more than 25 per cent the area of the section shall be increased to reduce the actual stresses to within the prescribed limit. Wind load stresses need not be combined with breaking load stresses.

218. **Reversal of Stress.**—Members subject to a reversal of stress due to a combination of dead load stresses and working load stresses shall be designed to take both tension and compression, each stress being increased by one half the smaller of the two stresses. Members subject to a reversal of stress due to wind stress combined with dead load stresses and working load stresses, or breaking load stresses combined with dead load stresses shall be designed to carry both stresses.

EQUIPMENT.

219. **Skips and Cages.**—Skips and cages shall be made of structural steel, as shown on the detail drawings. They shall be provided with guide shoes and safety devices. For inclined shafts the wheels shall have phosphor bronze bushings.

220. **Safety Detaching Hooks.**—All skips and cages shall be provided with effective detaching hooks. The case shall be designed to take the stress due to a loaded cage or skip dropping a vertical distance of two feet.

221. **Bin Gates.**—Unless otherwise specified all bin gates shall be of the undercut type. All gates shall be equipped with operating mechanism so that they can be opened in service by one man.

222. **Screens.**—Fixed screens shall be made of bars as shown on the drawings and shall be supported so that the bars will not be permanently deflected under the load. The screen bars shall be placed at an angle so that they will screen the ore or coal without choking up.

223. Shaking screens shall be carried on rollers and be driven by eccentric connecting bars. They shall be placed at proper slopes, and shall be provided with all necessary gates. Unless otherwise specified the screens shall be made of structural steel.

224. Rotary screens shall be made of structural and machinery steel, and shall perform the work required by the specifications.

225. **Coal Tipples or Dumps.**—Coal tipples or dumps shall be provided as shown on the detail plans or called for in the specifications.

226. **Dumping Devices.**—Where self-dumping skips or cages are used an efficient and satisfactory dumping device shall be provided.

227. **Head Sheaves.**—The head sheaves shall be substantial with the top flanges turned smooth and true to receive the hoisting rope. The sheave wheel shaft shall be of the best grade of machinery steel of ample strength, carefully and truly made. The sheave boxes shall be lined with the best quality of anti-friction metal and shall be adjustable to take up the wear. Unless otherwise specified the sheave wheels shall have wrought iron spokes.

228. **Landing Stage.**—An efficient landing device shall be furnished.

Details of Construction.

229. Unless otherwise provided for the details of construction are to be the same as for steel frame buildings.

230. **Design.**—In designing head frames, coal tipples, coal washers and breakers and similar structures care shall be used to strongly brace the different parts of the structure in order that it may be rigid. Preference shall be given to types of structures that are statically determinate. Where 4-post head frames and other statically indeterminate structures are used the stresses shall be calculated by taking account of the deformation and distortions of the members.* All bracing is to be made of stiff members; the use of rods or bars will not be permitted, except for sag rods and anchors. It is very important that head frames, coal tipples, coal washers and breakers and similar structures be made very rigid.

231. **Lengths of Compression Members.**—The length of compression members in head frames and shaker structures shall not exceed 100 times the least radius of gyration for main members nor 140 times the least radius of gyration for secondary bracing.

232. **Lengths of Tension Members.**—The length of tension members in head frames shall not exceed 150 times the least radius of gyration for main members, nor 200 times the least radius of gyration for secondary bracing. The length of a tension member is to be taken as the distance center to center of end connections.

233. **Splices.**—All splices in main members shall be designed to carry the full strength of the member.

234. **Reaming.**—The rivet holes for all field splices shall be punched to a diameter $\frac{3}{16}$ in. less than the finished hole and shall be reamed to the required size with the members bolted in place with an iron templet. All metal more than $\frac{5}{8}$ in. thick shall be punched and reamed, or be drilled from the solid.

235. **Minimum Thickness of Metal.**—The minimum thickness of metal in plates and sections shall be $\frac{5}{16}$ in., except for fillers.

236. **Erection.**—All field connections shall be riveted. Before the riveting is begun all field connections shall be fully drawn up with field bolts, in not less than one-half the holes of each joint.

237. **Materials and Workmanship.**—All materials and workmanship shall comply with the Specifications for Steel Frame Buildings unless otherwise specified.

238. **Painting.**—All steel work shall receive one coat of satisfactory graphite or carbon paint at the shop. Before erecting all abraded spots shall be touched up, and all rivet heads shall be painted as soon as accepted by the inspector. After the erection is complete all structural steel work shall be given two coats of satisfactory graphite or carbon paint. The three coats of paint shall be of different colors.

REFERENCES.—For additional data for the design of head frames, rock houses, coal tipples and other mine structures, and for numerous examples of structures, see the author's "The Design of Mine Structures." This book gives the calculation of stresses in head frames, and also gives a full discussion of the details of design of mine structures, including specifications, methods of construction and costs.

* For the calculation of the stresses in mine structures, see the author's "The Design of Mine Structures."

CHAPTER XI.

STEEL STAND-PIPES AND ELEVATED TANKS ON TOWERS.

DATA FOR DESIGN.—The following data will be of assistance in the design of steel stand-pipes and elevated tanks on towers. For definitions of stand-pipes and elevated tanks on towers, see the specifications in the latter part of this chapter.

Notation:—

h = distance in ft. of any point below the top of the stand-pipe or elevated tank;
d = diameter of the stand-pipe or elevated tank in feet;
r = radius of the stand-pipe or elevated tank in feet;
t = thickness of the shell in inches at any given point;
p = hydrostatic pressure in lb. per sq. in. at any point = $0.434h$;
S = stress per vertical lineal inch of stand-pipe;
s = unit stress in lb. per sq. in. in vertical section of stand-pipe;
S' = stress per horizontal lineal inch of stand-pipe;
s' = unit stress in lb. per sq. in. in horizontal section of stand-pipe;
S'' = stress per lineal inch along a circumferential line, due to wind;
s'' = unit stress in lb. per sq. in. in circumferential line, due to wind.

Formulas for Stresses in Stand-Pipes.—The stress per lineal vertical inch of stand-pipe is

$$S = \frac{62.5h\cdot d}{2 \times 12} = 2.6h\cdot d \tag{1}$$

The stress per sq. in. is

$$s = 2.6h\cdot d/t \tag{2}$$

The stress per horizontal lineal inch of stand-pipe due to the weight of stand-pipe W, is

$$S' = W/(12\pi\cdot d) = 0.026W/d \tag{3}$$

The stress per sq. in. is

$$s' = 0.026W/(d\cdot t) \tag{4}$$

For ordinary conditions the wind pressure is taken at 30 lb. per sq. ft. acting on two-thirds of the surface, or 20 lb. per sq. ft. on the entire surface; while for exposed positions the wind pressure may need to be taken as high as 45 lb. per sq. ft. acting on two-thirds of the surface, or 30 lb. per sq. ft. on the entire surface. Recent Prussian specifications require that circular chimneys be designed for two-thirds of 25 lb. per sq. ft. At 30 lb. per sq. ft. acting on two-thirds of the surface (20 lb. per sq. ft.) the bending moment at any distance h below the top, due to wind is

$$M = 20 \times d\cdot h \times h \times 12/2 = 120d\cdot h^2 \tag{5}$$

where M is in in.-lb.

The stress in the extreme fiber of the shell is

$$s'' = M\cdot y/I \tag{6}$$

Now $y = 12r$, $I = \frac{1}{4}\pi(r_1^4 - r_2^4) = t\cdot\pi\cdot r^3$ (approx.—r is in ft.^{3} and t in in.) $= t\cdot\pi\cdot r^3\cdot 12^3$ (in in.^{4}).

Substituting y and I in (6)

$$s'' = \frac{120d\cdot h^2\cdot r\cdot 12}{t\cdot\pi\cdot r^3\cdot 12^3}$$

$$= 1.06h^2/(t\cdot d) \tag{7}$$

The stress per lineal inch will be

$$S'' = 1.06h^2/d \tag{8}$$

If the allowable stress in the net section of the plate is 12,000 lb. per sq. in., and e = efficiency of joint, then from (2)

$$t = 2.6h \cdot d/(12{,}000 \times e) \tag{9}$$

where values of e for different conditions are given in Table IIa.

Formulas for Stresses in Elevated Steel Tanks.—The stress per lineal vertical inch of plate is the same as in stand-pipes

$$S = 2.6h \cdot d \tag{1}$$

and the unit stress in vertical joints is

$$s = 2.6h \cdot d/t \tag{2}$$

Stresses on Radial Joints.—Spherical Bottoms.—In a hemispherical bottom the radial stress per sq. in., T_1, will be one-half the stresses in a cylinder of the same radius and the same internal pressure.

$$T_1 = 2.6h \cdot d/(2t) = 2.6h \cdot r/t \tag{10}$$

In a segmental bottom (*b*) Fig. 1, the stress T_1' will be

$$T_1' = \frac{W \cdot \csc\theta}{2 \times 12\pi \cdot b \cdot t} = \frac{W \cdot \csc^2\theta}{24\pi \cdot r_1 \cdot t} \tag{11}$$

Now $W = 62.5h \cdot \pi \cdot b^2 = 62.5h \cdot \pi \cdot r_1^2 \cdot \sin^2\theta$, and

$$T_1' = \frac{62.5h \cdot r_1}{24t} = 2.6h \cdot r_1/t \tag{12}$$

which reduces to equation (10) for a hemispherical bottom when $r_1 = r$.

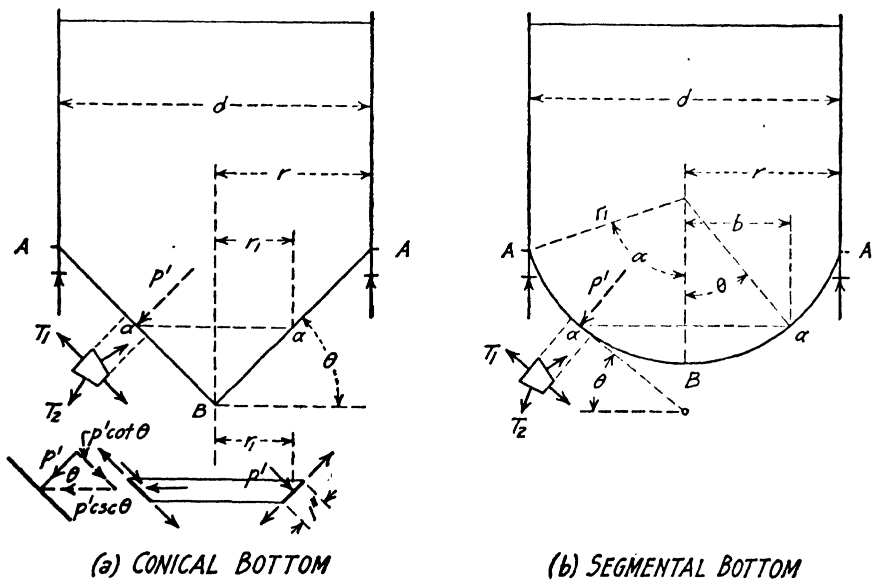


FIG. 1.

Stresses on Radial Joints. Conical Bottoms.—In a conical bottom the stress per sq. in. T_1'' will be from (*a*) Fig. 1,

$$T_1'' = \frac{W \cdot \csc \theta}{2r_1 \cdot \pi \cdot 12t} \tag{13}$$

Now

$$W = 62.5h \cdot \pi \cdot r_1^2,$$

and

$$T_1'' = \frac{62.5h \cdot \pi \cdot r_1^2 \cdot \csc \theta}{24r_1 \cdot \pi \cdot t} \tag{14}$$

$$= 2.6h \cdot r_1 \cdot \csc \theta / t \tag{15}$$

Stresses on Circumferential Joints. Conical Bottoms.—In (*a*) Fig. 1, pass two horizontal planes through the cone so that the intercept along the cone will be a unit in length. The tapered ring cut away has a pressure of p' lb. per lineal inch. This pressure p' may be resolved into a pressure along the element of the cone, $p_1 = p' \cot \theta$, and a horizontal pressure, $p_2 = p' \csc \theta$. The stress in circumferential joint will be

$$T_2'' = 12p_2 \cdot r_1/t = 12p' \cdot r_1 \cdot \csc \theta/t$$

$$= 12 \times 0.434h \cdot r_1 \cdot \csc \theta / t$$

$$= 5.2h \cdot r_1 \cdot \csc \theta / t \tag{16}$$

which is twice the stresses in the radial joints.

Stresses in Circumferential Joints.—Spherical Bottoms.—The radial unit stress in a hemispherical bottom is given by equation (12). Now in a segment of a spherical shell the curvature is the same in all directions, and the unit stress on a circumferential joint will be the same as on a radial joint, and

$$T_1' = T_2' = 2.6h \cdot r_1 / t \tag{17}$$

Connection Between Side and Bottom Plates.—With a conical bottom the inclined pull per lineal inch at the bottom of the circular tank will be from (15)

$$T_1''' = 2.6h \cdot r \csc \theta. \tag{18}$$

The compressive stress in the horizontal ring will be due to the horizontal components of the inclined stresses and will be

$$P' = T_1''' \cos \theta \cdot r \times 12$$

$$= 31.2h \cdot r^2 \cdot \cot \theta \tag{19}$$

There are no inclined or compressive stresses in a hemispherical bottom unless the circular shell and the hemispherical bottom are joined by an elliptical segment. If the radius of the circular tank divided by the radius of the segment = 2, there will be no secondary stresses (see "Stresses in Tank Bottoms," by Professor A. N. Talbot, The Technograph No. 16, p. 139).

Stresses in a Circular Girder.—The circular girder supports the weight of the tank, the contents of the tank, and its own weight. The load is uniformly distributed along the girder. The girder rests on or is supported by four or more columns, and transmits its load to them.

Let W = total load on girder in lb.;
r = radius of girder in in.;
n = number of posts;
$\alpha = 2\pi/n$ = angle at center subtended by radii through two consecutive posts:
α' = angle subtended at center by any arc;
M = direct bending moment in the girder at any point in in.-lb.;
T = torsional bending moment in girder at any point in in.-lb.;
S = shear in girder at any point in lb.;
$Pa = Pb$, etc., = reactions of columns in lb.

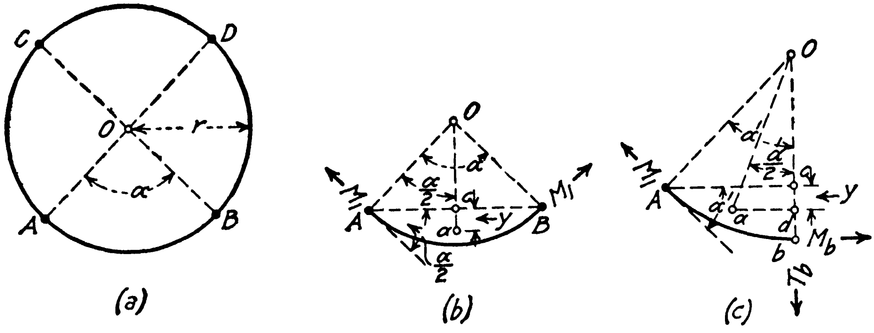


FIG. 2. CIRCULAR GIRDER.

Now in the author's "Design of Walls, Bins and Grain Elevators" it is proved that the bending moment at the supports is

$$M_1 = -\frac{W\cdot r}{n}\left(\frac{1}{\alpha} - \frac{1}{2}\cot\frac{\alpha}{2}\right) \tag{20}$$

and the maximum moment midway between the posts is

$$M_2 = M_1\cdot\cos\frac{\alpha}{2} + \frac{W\cdot r}{2n}\left(\sin\frac{\alpha}{2} - \frac{2\sin^2\frac{\alpha}{4}}{\frac{\alpha}{2}}\right) \tag{21}$$

The torsional moment is zero at the supports and midway between the columns, and is a maximum at the points of zero bending moment at points between the columns.

The torsional moment is

$$T_b = M_1\cdot\sin\alpha' - \frac{W\cdot r}{2n}.(1 - \cos\alpha') + \frac{W\cdot\alpha'\cdot r}{4}\left(1 - \frac{\sin\alpha'}{\alpha'}\right) \tag{22}$$

Values of M and T are given in Table Ia.

TABLE Ia.

STRESSES IN CIRCULAR GIRDERS.

No. of Posts.	Load on Post, Lb.	Max. Shear, Lb.	Bending Moment at Posts, In-lb.	Bending Moment Midway Between Posts, In-lb.	Angular Distance from Post to Point of Max. Torsion.	Max. Torsional Moment, In-lb.
4	$W \div 4$	$W \div 8$	$-0.03415W\cdot r$	$+0.01762W\cdot r$	19° 12′	$0.0053\ W\cdot r$
6	$W \div 6$	$W \div 12$	$-0.01482W\cdot r$	$+0.00751W\cdot r$	12 44	$0.00151\ W\cdot r$
8	$W \div 8$	$W \div 16$	$-0.00827W\cdot r$	$+0.00416W\cdot r$	9 33	$0.00063\ W\cdot r$
12	$W \div 12$	$W \div 24$	$-0.00365W\cdot r$	$+0.00190W\cdot r$	6 21	$0.000185W\cdot r$

Stresses in Columns.—The stresses in the columns will be due to the dead load and to the wind moment. The vertical components of the dead load stress will be equal to W divided by the number of columns, where W is the total weight of tank and the water. To calculate the stresses due to wind moment in the columns proceed as follows: Calculate the wind force by multiplying the exposed surface by the wind pressure, and assume the wind force as acting through the center of gravity of the exposed surface. The pressure on circular tanks may be taken at two-thirds of 30 lb. per sq. ft. of the surface at right angles to the direction of the wind. To calculate the stresses in the columns at any point pass a horizontal section through the columns

as in Fig. 3. Then the maximum vertical stress in column 1 will occur on the leeward side when the wind is blowing in the direction 1–1. If M is the wind moment about the axis $A-B$, the moment of the stresses in the column about axis $A-B$ will be equal to M. In a tower with 8 columns as in Fig. 3 we have (stress 1) $\times\ 2r$ + (stress 2) $\times\ 4r\cdot\cos 45^\circ = M$.

But Stress 1 is to Stress 2 as r is to $r\cdot\cos 45^\circ$; and Stress 1 $(2r + 2r) = M$. Stress 1 $= M/4r$, and Stress 2 $= 0.7M/4r$. In a 6 column tower the stress in the most remote post is $M/3r$ and in each of the others is $\frac{1}{2}\ M/3r$. In a 4 column tower the stress in each column is $M/2r$. If the columns are vertical the maximum stresses will occur at the foot of the columns; if the columns are inclined the stress should be calculated at both the top and the bottom. The maximum stresses will be the sum of the dead and wind load stresses.

Having calculated the vertical components of the stresses in the columns, the stress in the column will be equal to the vertical component multiplied by the secant of the angle between the column and a vertical line.

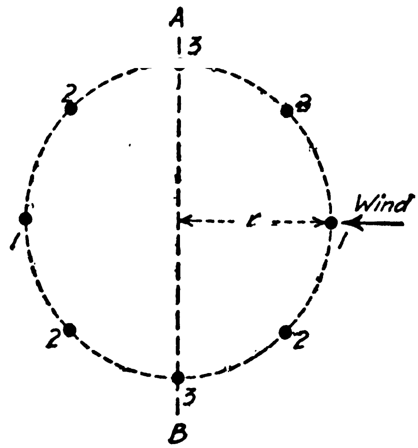


FIG. 3.

If the upward pull of the columns on the windward side is greater than the dead load when the tank is empty the column must be anchored down. The masonry footing should have a weight equal to at least one and one-half times the resultant upward pull.

DETAILS OF STEEL TANKS.—The standard plans in Fig. 10 and Fig. 11 and the Jackson, Minn., tank in Fig. 6, show the plates in alternate courses of different diameters, while the standard details of the Chicago Bridge and Iron Co. in Fig. 8 shows the plates telescoped with the edge of the plate for caulking on the inside so that it may be caulked from above. The standard specifications given in the last part of this chapter, also the specifications of the American Railway Engineering Association in the last part of this chapter both require that the plates in alternate courses be of different diameters as shown in Fig. 10, Fig. 11, and Fig. 6.

Hemispherical or segmental bottoms are now quite generally used, the conical bottom being rarely used on account of the difficulty in making a satisfactory connection to the tank cylinder. Spherical tank bottoms are used to a limited extent.

The standard details of the Chicago Bridge and Iron Co. for circular water tanks and hemispherical bottoms are given in Fig. 8, and the standard column details are shown in Fig. 9.

The properties for water tight joints together with shearing and bearing values of rivets are given in Table IIa. Standard plans for a 95,000 gallon tank on a 100 ft. tower are given in Fig. 10; while standard plans for a stand-pipe 20 ft. in diameter and 90 ft. high are given in Fig. 11. Table IIa and Fig. 10 and Fig. 11 were prepared by Mr. C. W. Birch-Nord to accompany the standard specifications printed in Trans. Am. Soc. C. E., Vol. 64, and partially reprinted in this chapter.

TABLE IIa.

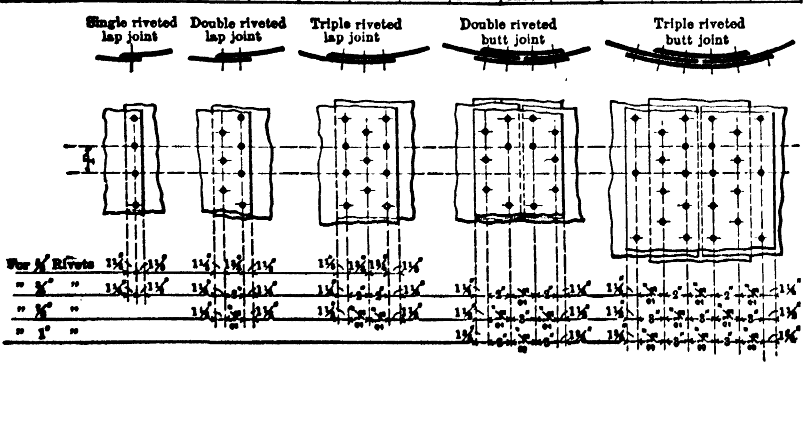
PROPERTIES OF WATERTIGHT JOINTS.

	Thickness of plate	Number of rows of rivets	5/8″ Rivets: Efficiency of joints in per cent	5/8″ Rivets: Pitch of rivets in inches	5/8″ Rivets: Effective section of plates	3/4″ Rivets: Efficiency of joints in per cent	3/4″ Rivets: Pitch of rivets in inches	3/4″ Rivets: Effective section of plates	7/8″ Rivets: Efficiency of joints in per cent	7/8″ Rivets: Pitch of rivets in inches	7/8″ Rivets: Effective section of plates	1″ Rivets: Efficiency of joints in per cent	1″ Rivets: Pitch of rivets in inches	1″ Rivets: Effective section of plates
Lap Joints	1/4″	1	48.7	1⅞	0.121									
		2	**70.7**	**2⅜**	**0.177**									
	5/16	1	39.5	1⅞	0.124	47.1	2⅛	0.147						
		2	65.4	2½	0.205	**70.5**	**3**	**0.220**						
	3/8	2	61.3	2	0.230	66.6	2⅝	0.250	**70.7**	**3⅛**	**0.265**			
		3	70.8	2⅜	0.265	75.6	3⅜	0.234	73.2	3⅜	0.274			
	7/16	2				63.5	2⅝	0.279	66.5	3	0.291			
		3				72.3	3⅛	0.317	**75.2**	**4**	**0.329**			
	1/2	2				58.9	2⅜	0.295	63.8	2⅞	0.319			
		3				69.4	2⅝	0.347	**72.6**	**3⅜**	**0.363**			
	9/16	2							61.0	2⅞	0.344			
		3							70.5	3⅜	0.397			
Butt Joints	7/16	2				72.0	3⅛	0.315	72.3	3⅜	0.316			
		3				82.2	3⅛	0.359	84.7	3⅜	0.370			
	1/2	2				72.0	3⅛	0.360	72.3	3⅜	0.362			
		3				80.8	3⅛	0.405	82.8	3⅜	0.415			
	9/16	2				72.0	3⅛	0.405	**72.3**	**3⅜**	**0.407**			
		3				80.5	3⅛	0.453	82.1	3⅜	0.463			
	5/8	2				70.7	3	0.442	**72.3**	**3⅜**	**0.452**			
		3				78.4	3	0.490	81.0	3⅜	0.506			
	11/16	2				68.3	2⅞	0.469	**72.3**	**3⅜**	**0.498**			
		3				75.7	2⅞	0.522	80.3	3½	0.552			
	3/4	2				66.4	2¾	0.498	**70.2**	**3⅜**	**0.526**			
		3				73.8	2¾	0.553	78.0	3⅜	0.585			
	13/16	2							68.3	3½	0.553			
		3							**75.5**	**3½**	**0.611**			
	7/8	2							66.5	3	0.582			
		3							**74.1**	**3**	**0.647**			
	15/16	2	Note:									70.1	3½	0.657
		3	**Heavy figures indicate**									**76.5**	**3⅜**	**0.717**
	1″	2	**economical riveted joints**									67.3	3½	0.673
		3										**74.7**	**3½**	**0.747**

Note: The distances between rivets at caulked edges shall never exceed 10 times the thickness of plates or straps. The thickness of each strap for butt joints shall never be less than half the thickness of the plates plus $\frac{1}{16}$ inch.

SHEARING AND BEARING VALUE OF RIVETS.

Diameter of rivets, in inches	Area in square inches	Single Shear at 9000 lb. per sq. in.	Bearing value for different thicknesses of plates, in inches, at 18000 lb. per sq. in.: 1/4″	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
5/8	0.3068	2761	2813	3516	4219	4922	5625	6328	7031						
3/4	0.4418	3976	3375	4219	5063	5906	6750	7594	8438	9281	10125				
7/8	0.6013	5412	3938	4922	5906	6891	7875	8859	9844	10828	11813	12797	13781		
1	0.7854	7069	4500	5625	6750	7875	9000	10125	11250	12375	13500	14625	15750	16875	18000



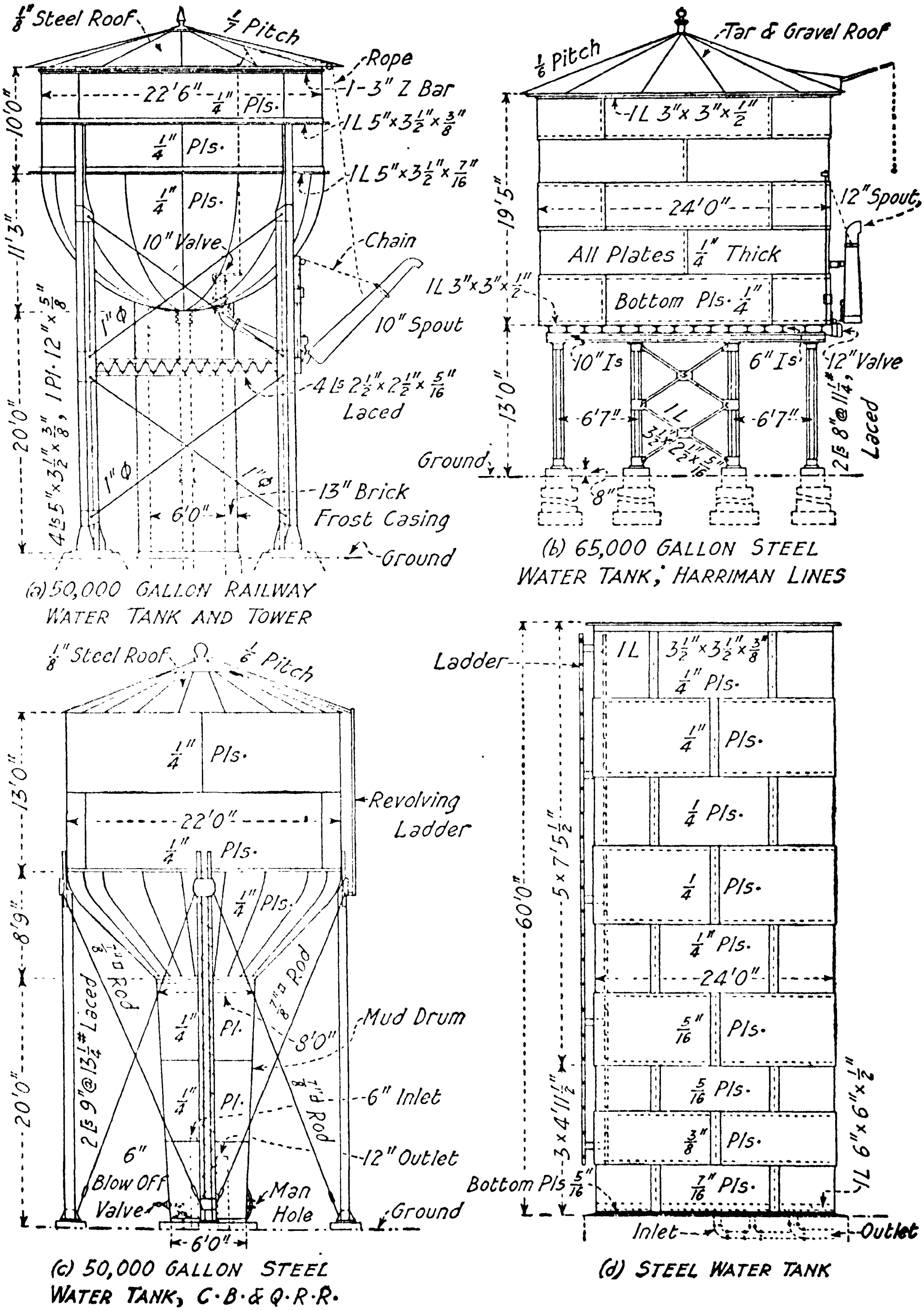


FIG. 4. TYPICAL STEEL WATER TANKS.

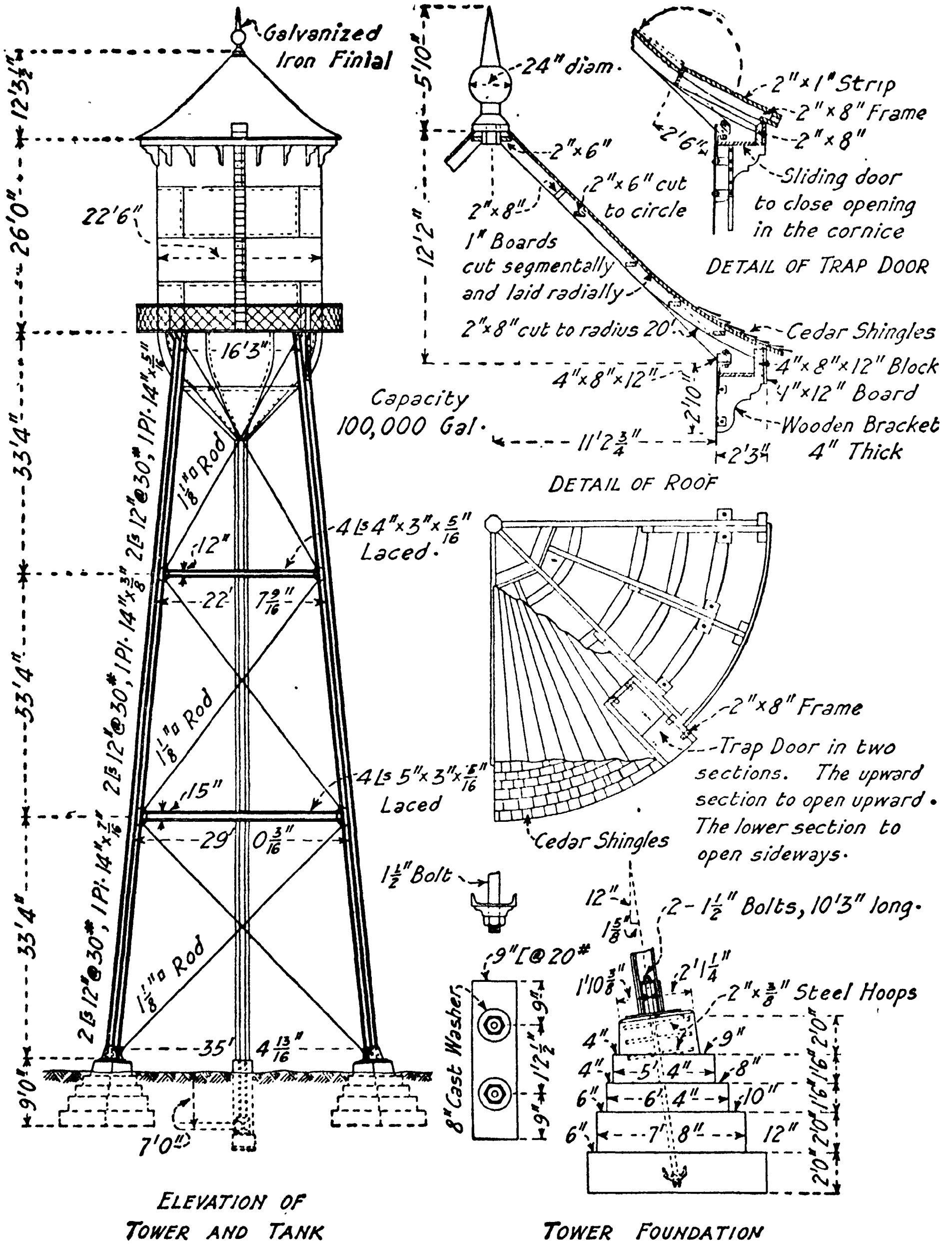


FIG. 5. ELEVATED TANK AND TOWER, JACKSON, MINN.

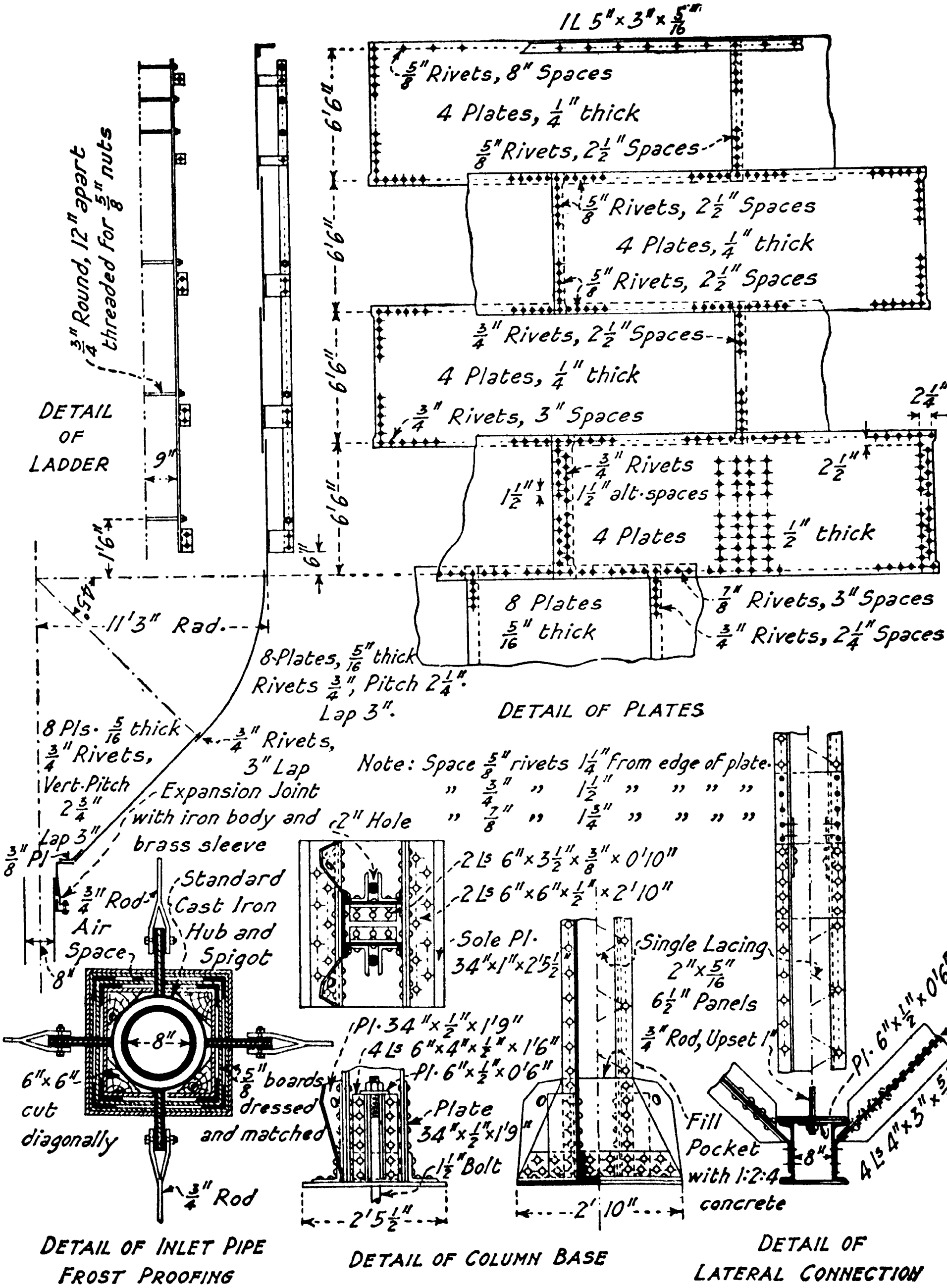


FIG. 6. ELEVATED TANK AND TOWER, JACKSON, MINN.

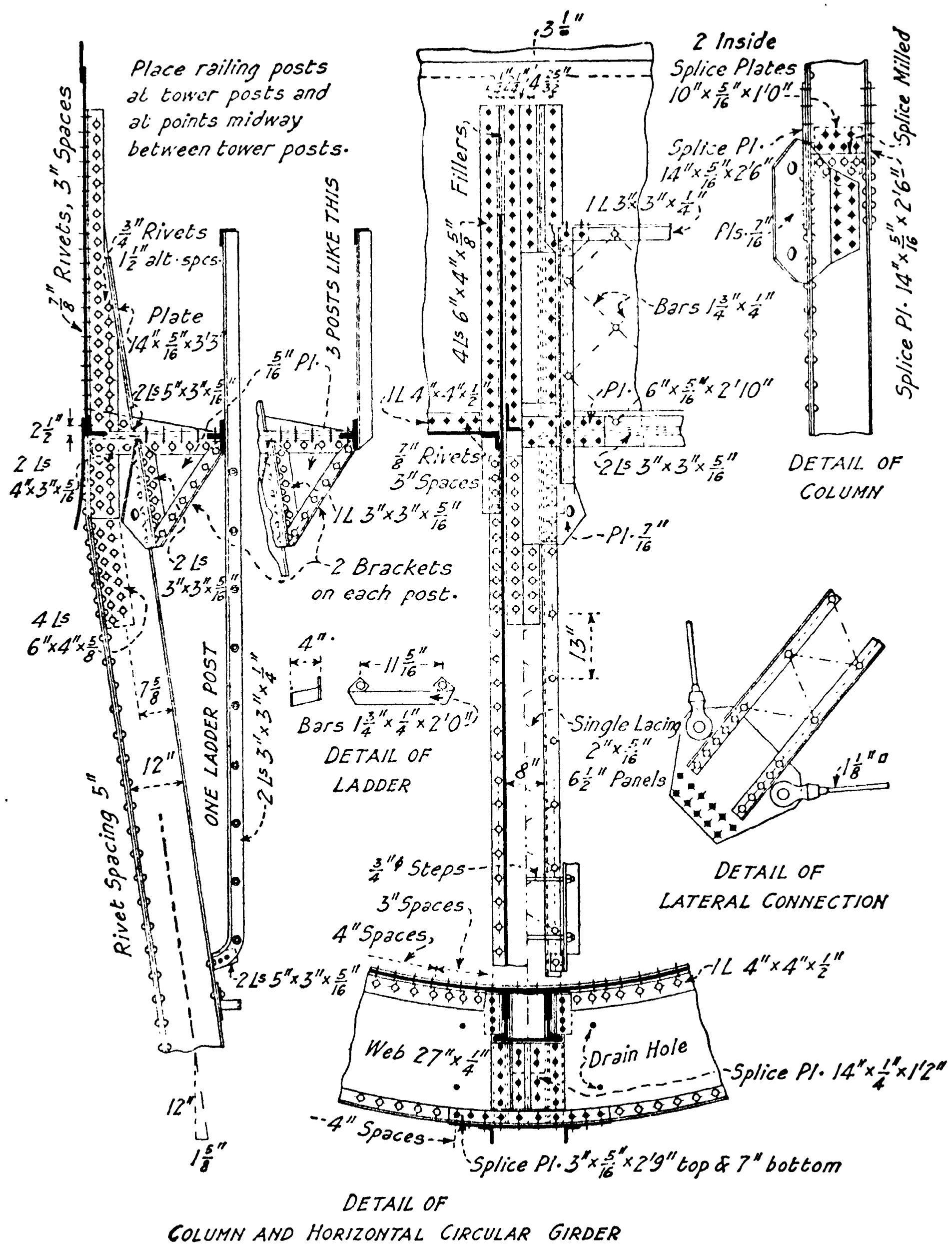


FIG. 7. ELEVATED TANK AND TOWER, JACKSON, MINN.

DETAILS OF STEEL TOWERS.—Steel towers are commonly made with four columns, although eight or twelve columns are sometimes used for large elevated tanks. The columns of towers are commonly made of two channels, laced top and bottom; of two channels with top cover plate and bottom lacing; of a built *H* section made of plates and angles, or a rolled *H* section. Z-bars are now very difficult to obtain and the Z-bar column should not be used. The struts are made of built channels, or of angles, or of plates and angles. The diagonal bracing is commonly made of rods with adjustable clevises or turnbuckles.

EXAMPLES OF STEEL STAND-PIPES AND ELEVATED TANKS ON TOWERS.—The design of steel stand-pipes and elevated tanks on towers will be illustrated by describing several typical examples.

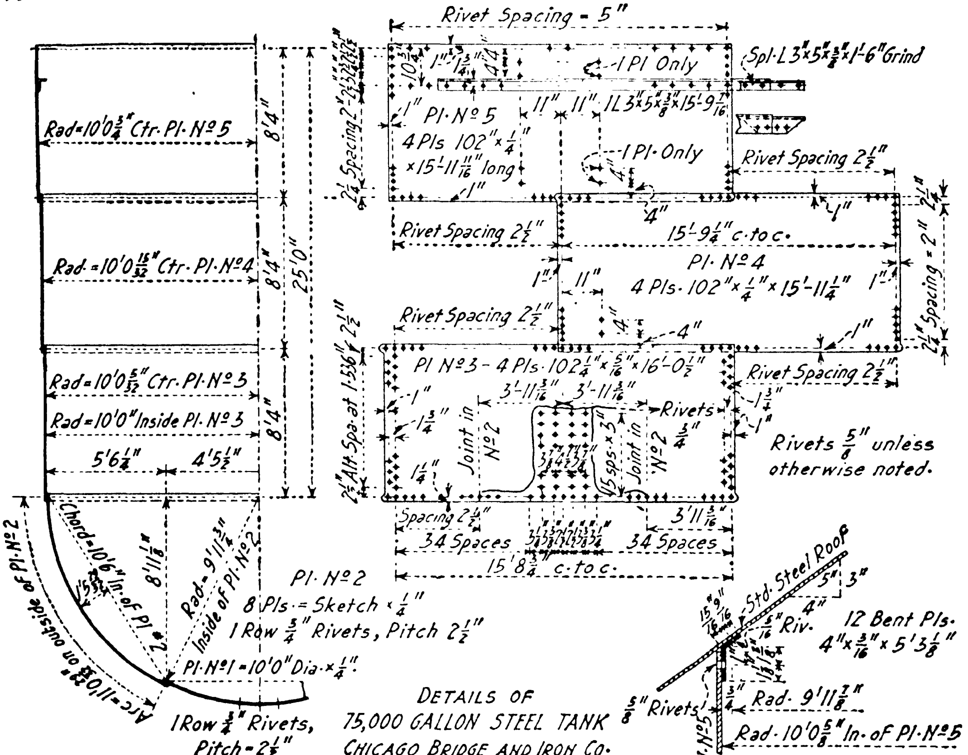


FIG. 8. DETAILS OF TANK AND HEMISPHERICAL BOTTOM. CHICAGO BRIDGE & IRON CO.

Railway Water Tanks.—Four typical examples of steel water tanks are shown in Fig. 4; the 50,000 gallon railway water tank in (*a*) Fig. 4 was designed by the American Bridge Company; the 65,000 gallon water tank in (*b*) is a standard tank on the Harriman Lines; the 50,000 gallon tank in (*c*) was designed by the C. B. & Q. R. R.; while (*d*) is a typical stand-pipe.

Elevated Tank and Tower for Jackson, Minn.—Details of the steel elevated tank and tower designed by Mr. L. P. Wolff, Consulting Engineer, St. Paul, Minn., for Jackson, Minn., are shown in Fig. 5, Fig. 6, and Fig. 7. A general plan and details of the foundations and the roof are shown in Fig. 5. Details of the riveting of the tank plates; details of the columns, and details of the frost proofing are shown in Fig. 6. Details of the circular girder, and the connections of the columns are shown in Fig. 7. The tank has a hemispherical bottom with a conical sub-bottom.

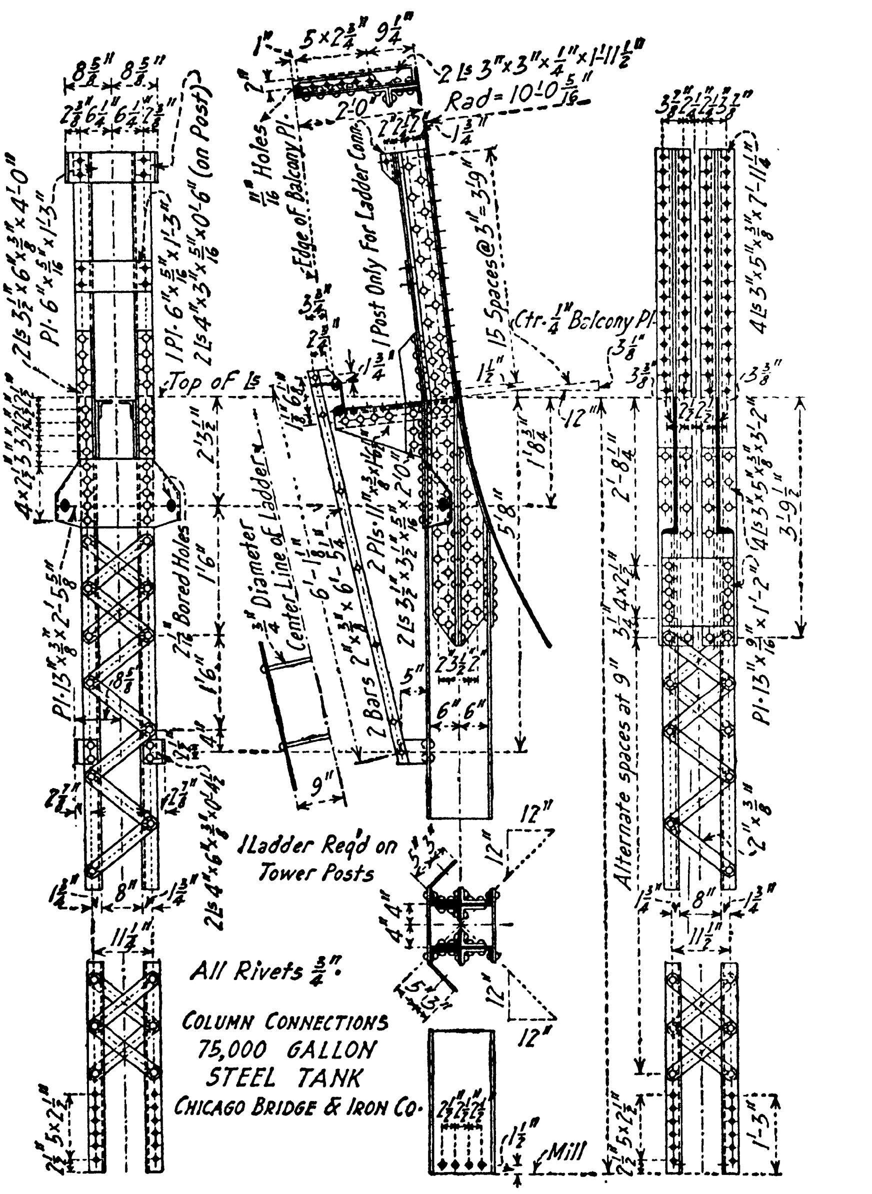


Fig. 9. Details of Column Connections for Elevated Tank and Tower. Chicago Bridge & Iron Co.

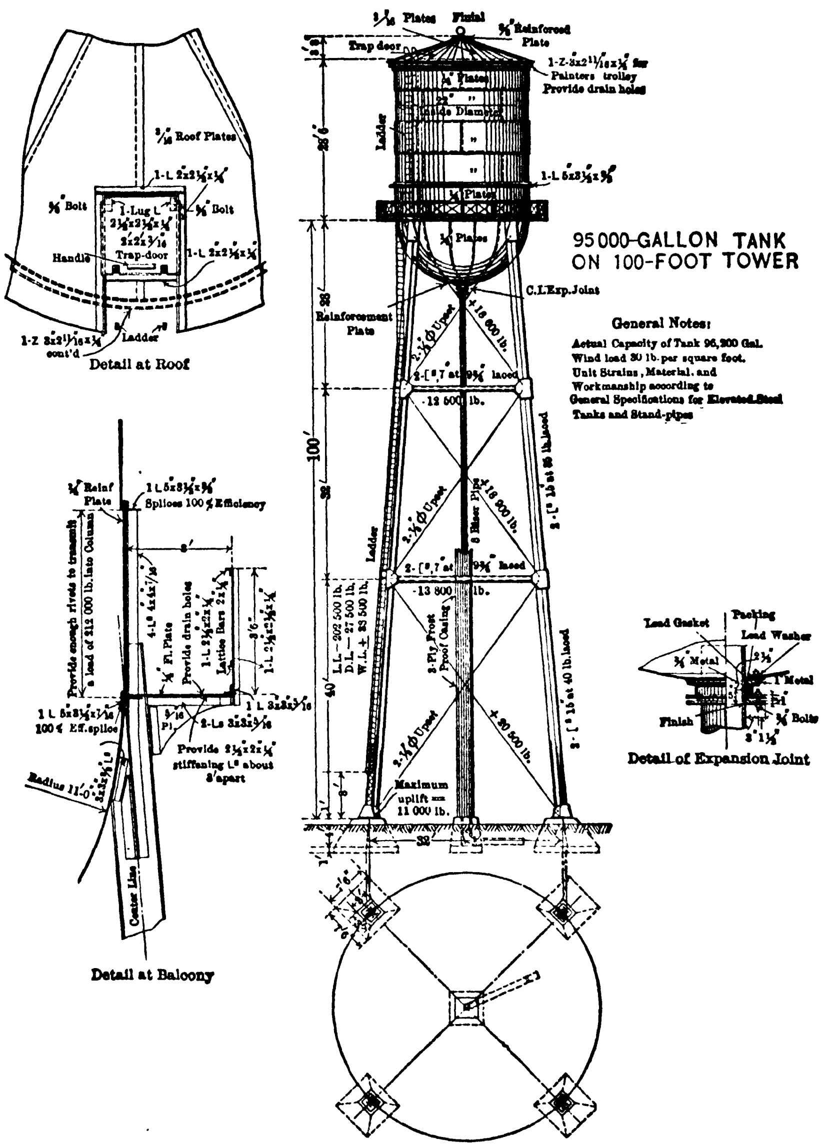


FIG. 10. STANDARD PLAN OF ELEVATED TANK ON TOWER, BY C. W. BIRCH-NORD.
(Trans. Am. Soc. C. E., Vol. 64, 1909.)

The details work out very satisfactorily. Mr. Wolff has designed a number of elevated tanks and towers following the standard details in the Jackson tank. The details of construction are shown by the drawings.

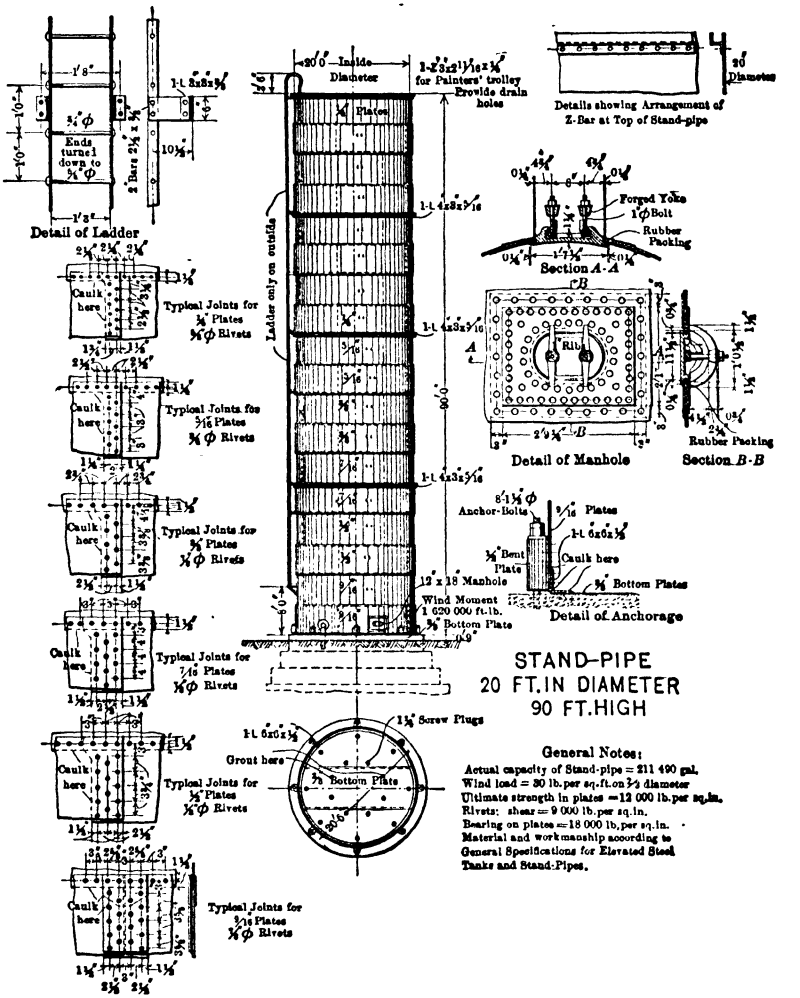


FIG. 11. STANDARD PLAN OF STAND-PIPE, BY C. W. BIRCH-NORD. (Trans. Am. Soc. C. E., Vol. 64, 1909.)

SPECIFICATIONS.—The details of design of steel stand-pipes and elevated tanks on towers are given in the specifications prepared by Mr. C. W. Birch-Nord and the specifications of the American Railway Engineering Association. Both of these specifications are printed in the last part of this chapter.

GENERAL SPECIFICATIONS FOR ELEVATED STEEL TANKS ON TOWERS, AND FOR STAND-PIPES.*

PART I. DESIGN OF ELEVATED STEEL TANKS ON TOWERS.

Definition.—1. An elevated tank is a vessel placed on a tower in order to furnish a certain required pressure head. The tank is filled through a riser or inlet pipe.

2. Elevated tanks are mostly used in connection with pumping stations, or are connected directly to Artesian wells, in order to store water under pressure.

3. As practically all tanks are cylindrical, this specification will only have reference to those of that shape.

Loads.—4. The dead load shall consist of the weight of the structural and ornamental steelwork, platforms, roof construction, piping, etc.

5. The live load shall be the contents of the tank, the movable load on the platforms and roof, and the wind pressure.

6. The live load on the platforms and roof shall be assumed at 30 lb. per sq. ft., or a 200-lb. concentrated load applied at any point.

7. The wind pressure shall be assumed at 30 lb. per sq. ft., acting in any direction. The surfaces of cylindrical tanks exposed to the wind shall be calculated at two-thirds of the diameter multiplied by the height. Similar assumptions may also be made for spherical and conical surfaces by using the correct heights.

8. The live load on platforms and roof shall not be considered as acting together with the wind pressure.

Unit Stresses.—9. All parts of the structure shall be proportioned so that the sum of the dead and live loads shall not cause the stresses to exceed those given in Table I.

TABLE I.

Tension in tank plates	12,000 lb. per sq. in. of net area.
Tension in other part of structure	16,000 lb. per sq. in. of net area.
Compression	16,000 lb. per sq. in. reduced.
Shear on shop rivets and pins	12,000 lb. per sq. in.
Shear on field rivets (tank rivets) and bolts	9,000 lb. per sq. in.
Shear in plates	10,000 lb. per sq. in. of gross area.
Bearing pressure on shop rivets and pins	24,000 lb. per sq. in.
Bearing pressure on field rivets (tank rivets)	18,000 lb. per sq. in.
Fiber strain in pins	24,000 lb. per sq. in.

10. For compression members, the permissible unit stress of 16,000 lb. shall be reduced by the formula:

$$p = 16{,}000 - 70\,l/r,$$

where p = permissible working stress in compression, in lb. per sq. in.:
l = length of member, from center to center of connections, in inches;
r = least radius of gyration of section, in inches.

The ratio, l/r, shall never exceed 120 for main members and 180 for struts and roof construction members.

11. Stresses due to wind may be neglected if they are less than 25 per cent of the combined dead and live loads.

12. Unit stresses in bracing and other members taking wind stresses may be increased to 20,000 lb. per sq. in., except as shown in Section 11.

13. The pressures given in Table II will be permissible on bearing plates.

TABLE II.

Brickwork with cement mortar	200 lb. per sq. in.
Portland cement concrete	350 lb. per sq. in.
First-class sandstone	400 lb. per sq. in.
First-class limestone	500 lb. per sq. in.
First-class granite	600 lb. per sq. in.

* Condensed from Specifications by C. W. Birch-Nord, Assoc. M. Am. Soc. C. E., Trans. Am. Soc. C. E., Vol. 64, pp. 548 to 563. The preliminary statement and the specifications for the foundations have been omitted. These specifications have been adopted by the American Bridge Company.

Details of Construction.—14. The plates forming the sides of cylindrical tanks shall be of different diameters, so that the courses shall lap over each other, inside and outside, alternately.

15. The joints for the horizontal seams, and for the radial seams in spherical bottoms, shall preferably be lap joints.

16. For vertical seams double-riveted lap joints shall be used for $\frac{1}{4}$, $\frac{5}{16}$, and $\frac{3}{8}$ in. plates. Triple lap joints shall be used for $\frac{7}{16}$ and $\frac{1}{2}$ in. plates; double-riveted butt joints shall be used for $\frac{9}{16}$, $\frac{5}{8}$, $\frac{11}{16}$ and $\frac{3}{4}$ in. plates; and triple-riveted butt joints for $\frac{13}{16}$, $\frac{7}{8}$, $\frac{15}{16}$ and 1 in. plates.

17. Rivets $\frac{5}{8}$ in. in diameter shall be used for $\frac{1}{4}$ in. plates; rivets $\frac{3}{4}$ in. in diameter shall be used for $\frac{5}{16}$ in. plates; rivets $\frac{7}{8}$ in. in diameter shall be used for $\frac{3}{8}$ to $\frac{7}{8}$ in. plates, inclusive. Rivets 1 in. in diameter shall be used for $\frac{15}{16}$ in. and 1 in. plates.

Rivets shall be spaced so as to make the most economical seams (70 to 75 per cent efficiency). A table of riveted joints is given in Table IIa.

18. In no case shall the spacing between rivets along the caulked edges of plates be more than ten times the thickness of the plates. All rivets shall be entered from the inside of the tank, and shall be driven from the outside, that is, new heads on rivets shall always be formed from the opposite side of the plate on which the caulking is done.

19. Plates $\frac{5}{8}$ in. thick, and not more than $\frac{7}{8}$ in. thick, shall be sub-punched with a punch $\frac{3}{16}$ in. smaller in diameter than the nominal size of the rivets, and shall be reamed to a finished diameter not more than $\frac{1}{16}$ in. larger than the rivet. Plates thicker than $\frac{7}{8}$ in. shall be drilled.

20. The minimum thickness of the plates for the cylindrical part shall be $\frac{1}{4}$ in. The thickness of the plates in spherical bottoms shall never be less than that of the lower course in the cylindrical part of the tank.

21. The facilities at the plant where the material is to be fabricated will be investigated before the material is ordered.

22. All plates shall be sheared or planed to a proper bevel along the edges for caulking.

23. All plates shall be caulked along the beveled edges from the inside of the tank, and with a round-nosed tool. The use of foreign material for caulking, such as lead, copper, filings, cement, etc., will not be permitted.

24. The plates in tanks for the storage of oil shall be beveled on both sides for outside and inside caulking.

25. The radial sections of spherical bottoms shall be made in multiples of the number of columns supporting the tank, and shall be reinforced at the lower parts, where holes are made for piping.

26. When the center of the spherical bottom is above the point of connection with the cylindrical part of the tank, there shall be provided a girder at said point of connection to take the horizontal thrust. The horizontal girder may be made in connection with a balcony. This also applies where the tank is supported by inclined columns.

27. The balcony around the tank shall be 3 ft. wide, and shall have a floor-plate $\frac{1}{4}$ in. thick, which shall be punched for drainage. The balcony shall be provided with a suitable railing, 3 ft. 6 in. high.

28. The upper parts of spherical bottom plates shall always be connected on the inside of the cylindrical section of the tank.

29. In order to avoid eccentric loading on the tower columns, and local stresses in spherical bottoms, the connections between the columns and the sides of the tank shall be made in such a manner that the center of gravity of the column section intersects the center of connection between the spherical bottom and the sides of the tank. Enough rivets shall be provided above this intersection to transmit the total column load.

30. If the tank is supported on columns riveted directly to the sides, additional material shall be provided in the tank plates riveted directly to the columns to take the shear. The shear may be taken by providing thicker tank plates, or by reinforcement plates at the column connections, while bending moments shall be taken by upper and lower flange angles. Connections to columns shall be made in such a manner that the efficiency of the tank plates shall not be less than that of the vertical seams.

31. For high towers, the columns shall have a batter of 1 to 12. The height of the tower shall be the distance from the top of the masonry to the connection of the spherical bottom, or the flat bottom, with the cylindrical part of the tank.

32. Near the top of the tank there shall be provided one Z-bar to act as a support for the painter's trolley, and for stiffening the tank. Its section modulus shall not be less than $D^2/250$, where D is the diameter of the tank in feet. If the upper part of the tank is thoroughly held by the roof construction, this may be reduced.

33. On large tanks, circular stiffening angles shall be provided in order to prevent the plates from buckling during wind storms. The distance between the angles shall be determined by the formula:

$$d = 900\, t^{\frac{3}{2}}/D,$$

where d = approximate distance between angles, in feet;
t = thickness of tank plates, in inches;
D = diameter of tank, in feet.

34. The top of the tank will generally be covered with a conical roof of thin plates; and the pitch shall be 1 to 6. For tanks up to 22 ft. in diameter, the roof plates will be assumed to be self-supporting. If the diameter of the tank exceeds 22 ft., angle rafters shall be used to support the roof plates, which are generally $\frac{1}{8}$ in. thick.

Plates of the following thicknesses will be assumed to be self-supporting for various diameters:
$\frac{3}{32}$ in. plate, up to a diameter of 18 ft.
$\frac{1}{8}$ in. plate, up to a diameter of 20 ft.
$\frac{3}{16}$ in. plate, up to a diameter of 22 ft.

Rivets in the roof plates shall be from $\frac{1}{4}$ to $\frac{5}{16}$ in. in diameter, and shall be driven cold. These rivets need not be headed with a button set.

35. A trap-door, 2 ft. square, shall be provided in the roof plate. Near the top of the higher tanks, there shall be a platform with a railing, for the safety of the men operating the trap-door.

36. There shall be an ornamental finial at the top of the roof.

37. There shall be a ladder, 1 ft. 3 in. wide, extending from a point about 8 ft. above the foundation to the top of the tank, and also one on the inside of the tank. Each ladder shal' be made of two $2\frac{1}{2}$ by $\frac{3}{8}$ in. bars with $\frac{3}{4}$ in. round rungs 1 ft. apart. On large, high tanks, 30 ft. or more in diameter, a walk shall be provided from the column nearest the ladder to the expansion joint on the riser or inlet pipe.

38. In designing a tank, a height of 6 in. shall be added to the required height of the tank if an overflow pipe is not specified by the owner.

39. Each elevated tank shall be furnished with a riser or inlet pipe, the size of which shall be determined by the rate at which the tank must be filled. The size of the riser pipe will be specified by the owner. The outlet pipe, in most cases, is not required, as the riser or inlet pipe will serve the same purpose, but it shall be furnished if demanded by the owner.

40. All pipes entering the tank shall have cast-iron expansion joints with rubber packing, and facilities for tightening such joints. The expansion joint, generally, shall be fastened to the bottom of the tank with bolts having lead washers. The tank plates shall be reinforced where the pipes enter the tank.

41. All pipes entering the tank shall be thoroughly braced laterally with adjustable diagonal bracing at the panel points of the tower.

42. The diagonal bracing in the tower shall preferably be adjustable, and shall be calculated for an initial stress of 3,000 lb. in addition to wind stresses, etc.

43. The size and number of the anchor-bolts in the tower shall be determined by the maximum uplift when the tank is empty. The anchor-bolts in the tower, where the maximum uplift is greater than 10,000 lb., shall be fastened directly to the columns with bent plates or similar details. In all other cases it will be sufficient to connect the anchor-bolts directly to the base-plates.

The tension in anchor-bolts shall not exceed 15,000 lb. per sq. in. of net area. The minimum section shall be limited to a diameter of $1\frac{1}{4}$ in. The details shall be made so that the anchor-bolts will develop their full strength, and, at the lower end, they shall be furnished with an anchor-plate, not less than $\frac{1}{2}$ in. thick, to assure good anchorage to the foundation without depending on the adhesion between the concrete and the steel.

44. The concrete foundation shall be assumed to have a weight of 140 lb. per cu. ft., and shall be sufficient in quantity to take the uplift, with a factor of safety of $1\frac{1}{2}$.

45. Three-ply frost-proof casing shall be provided, if necessary, around the pipes leading to and from the tank. This casing shall be composed of two layers of $\frac{7}{8}$ by $2\frac{1}{2}$ in. dressed lumber, and each layer shall be covered with tar paper or tarred felt, and one outside layer of $\frac{7}{8}$ by $2\frac{1}{2}$ in. dressed and matched flooring. The lumber shall be in lengths of about 12 ft. There shall be a 1 in. air space between the layers of lumber, and wooden rings or separators shall be nailed to them every 3 ft. (In very cold climates it is good practice to fill the space between the pipes and the first layer of lumber with hay or similar material.) The frost casing may be square or cylindrical; it shall be braced to the tower with adjustable diagonal bracing, as described for pipes in Section 41.

46. All detailed drawings shall be subject to the owner's approval before work is commenced.

47. For materials, workmanship, inspection, painting, and testing, see Part III; for foundations, see Part IV.

PART II. DESIGN OF STAND-PIPES.

Definition.—1. A stand-pipe is a tank, generally cylindrical, used for the storage of water, oil, etc. Its height, in most cases, is considerably greater than its diameter; it has a flat bottom, and rests directly on its foundation.

2. Stand-pipes are economical only in special cases: where their capacity is more important than pressure, or where local conditions are such that an elevated tank is not required.

3. Stand-pipes for the storage of oil are an exception. These are generally of very large diameter, while the height may not exceed 40 ft.; they are usually referred to as tanks.

4. Stand-pipes are filled and emptied through pipes connected with their sides or bottom, and are provided with manholes for cleaning purposes.

5. In cold climates roofs are generally omitted on stand-pipes used for water supply, on account of the formation of ice. In warmer climates there may be roofs in order to prevent the water from becoming a breeding place for mosquitos, flies, etc. Stand-pipes used for the storage of oil or other fluids from which rain-water is to be excluded should always be roofed.

Loads.—6. The dead load shall consist of the weight of structural and ornamental steel work, and the roof construction, if any.

7. The live load shall be the contents of the stand-pipe, the movable load on the eventual roof, and the wind pressure.

8. The eventual live load on the roof shall be assumed at 30 lb. per sq. ft., or a 200 lb. concentrated load applied at any point.

9. The wind pressure shall be assumed at 30 lb. per sq. ft. acting in any direction. The surfaces of cylindrical stand-pipes exposed to the wind shall be calculated at two-thirds of the diameter multiplied by the height.

10. The eventual live load on the roof, if the stand-pipe is roofed, shall not be considered as acting together with the wind pressure.

Stresses.—11. All parts of the structure shall be porportioned so that the sum of the dead and live load stresses shall not exceed the stresses given in Table III.

TABLE III.

Tension in plates forming sides or bottom of stand-pipes......	12,000 lb. per sq. in. of net area.
Tension in roof construction..............................	16,000 lb. per sq. in. of net area.
Compression in roof construction..........................	16,000 lb. per sq. in. reduced
Shear on shop rivets in roof, etc.........................	12,000 lb. per sq. in.
Shear on field rivets (in stand-pipe plates) and bolts..........	9,000 lb. per sq. in.
Shear in plates...	10,000 lb. per sq. in.
Bearing pressure on shop rivets...........................	24,000 lb. per sq. in.
Bearing pressure on field rivets (in stand-pipe plates).........	18,000 lb. per sq. in.

12. For compression members in the roof construction, the permissible unit stress of 16,000 lb. shall be reduced by the formula:

$$p = 16{,}000 - 70\,l/r,$$

where p = permissible working stress in compression, in lb. per sq. in.;
l = length of member, from center to center of connections, in inches;
r = least radius of gyration of section, in inches. The ratio, l/r, shall never exceed 180.

13. Stresses due to wind may be neglected if they are less than 25 per cent of the combined dead and live loads.

14. The average permissible pressures on masonry shall be as given in Table II, Part I.

Details of Construction.—15. The plates forming the sides of the stand-pipe shall be of different diameters, so that the courses shall lap over each other, inside and outside, alternately.

16. The joints for the horizontal seams in the sides, and for the bottom plates, shall preferably be lap joints.

17. For further information regarding riveted joints, etc., see Part I, Sections 16, 17, 18, and 19.

18. The minimum thickness of the plates forming the sides shall be $\frac{1}{4}$ in. and $\frac{5}{16}$ in. for the bottom plates, except for oil tanks on a sand foundation. The bottom plates for ordinary stand-pipes shall be provided with tapped holes, $1\frac{1}{2}$ in. in diameter, with screw plugs, spaced at about 4 ft. centers, to permit of filling with cement grout on top of the foundation of the masonry while the bottom part is being erected, in order to secure proper bearing.

19. Oil tanks of large diameter are generally set directly on a sand foundation, and do not need any holes in the bottom plates for filling beneath with cement grout. In such cases, $\frac{1}{4}$ in. bottom plates will be sufficient.

20. The bottom plates shall be connected with the sides by an angle iron riveted inside the stand-pipe. This angle iron shall be bevel sheared for caulking along both legs. For the caulking of plates, see Part I, Sections 22 and 23.

21. On the side and near the bottom there shall be a 12 by 18 in. manhole of elliptical shape. In the same manner, or on the bottom plates, flanges shall be provided for the connection of

inlet and outlet pipes of the sizes specified by the owner. All openings in stand-pipes shall be properly reinforced by forged rings or plates.

22. For stiffening angles, etc., see Part I, Sections 32 and 33.

23. In cases where a roof is used see Section 5; Sections 34, 35, and 36 of Part I should also be followed.

24. There shall be an outside ladder, 1 ft. 3 in. wide, extending from a point about 8 ft. above the foundation to the top of the stand-pipe. The ladder shall be made of two $2\frac{1}{2}$ by $\frac{3}{8}$ in. bars with $\frac{3}{4}$ in. round rungs 1 ft. apart. An inside ladder will not be required. (In no case should inside ladders be provided on stand-pipes in climates where ice will form. Owners of oil tanks often specify stairways to take the place of ladders.) All ladders shall be able to sustain a concentrated load of at least 800 lb.

25. Large stand-pipes for oil storage, the heights of which are very small compared with their diameter, will generally be set directly on a sand foundation, and will not need any anchorage whatever, as the overturning moment is very small in comparison with the resisting moment.

26. Stand-pipes of the ordinary type, for water storage, shall be set on concrete foundations, and shall be anchored thoroughly thereto with anchor-bolts not less than $1\frac{1}{4}$ in. in diameter, set deep enough to take the necessary uplift, and provided with an anchor plate not less than $\frac{1}{2}$ in. thick in the masonry. All anchor bolts shall be connected directly to the sides of the stand-pipe with bent plates or similar details. The unit stress in anchor-bolts shall not exceed 15,000 lb. per sq. in. of net area. See Part I, Section 43.

27. All detailed drawings shall be subject to the owner's approval before work is commenced.

28. For materials, workmanship, inspection, painting, and testing, see Part III; for foundations, see Part IV.

Part III. Materials, Workmanship, Inspection, Painting, and Testing.

Structural Steel.—1. The steel shall be made by the open-hearth process.

2. The chemical and physical properties shall conform to the following limits:

Elements considered.	Structural Steel.	Rivet Steel.
Phosphorus, maximum { Basic	0.04 per cent	0.04 per cent
Phosphorus, maximum { Acid	0.06 " "	0.04 " "
Sulphur, maximum	0.05 " "	0.04 " "
	Desired	Desired
Ultimate tensile strength, in pounds per square inch	60,000	50,000
Elongation: minimum percentage in 8 in. Fig. 1	$\frac{1,500,000}{\text{Ultimate tensile strength}}$	$\frac{1,500,000}{\text{Ultimate tensile strength}}$
Elongation: minimum percentage in 2 in. Fig. 2	22	..
Character of fracture	Silky	Silky
Cold bends without fracture	180° flat	180° flat

The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

3. If the ultimate strength varies more than 4,000 lb. from that desired, a re-test shall be made on the same gage, which to be acceptable, shall be within 5,000 lb. of the desired ultimate.

4. Chemical determination of the percentages of carbon, phosphorus, sulphur, and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for by the purchaser, in which case an excess of 25 per cent above the required limits will be allowed.

5. Specimens for tensile and bending tests, for plates, shapes, and bars, shall be made by cutting coupons from the finished product, which shall have both faces rolled and both edges milled to the form shown by Fig. 1; or with edges parallel; or they may be turned to a diameter of $\frac{3}{4}$ in. for a length of at least 9 in. with enlarged ends.

6. Rivet rods shall be tested as rolled.

7. Specimens shall be cut from the finished rolled or forged bar, in such manner that the center of the specimen shall be 1 in. from the surface of the bar. The specimen for the tensile test shall be turned to the form shown by Fig. 2. The specimen for the bending test shall be 1 in. by $\frac{1}{2}$ in. in section.

8. Material which is to be used without annealing or further treatment shall be tested in the condition in which it comes from the rolls. When material is to be annealed, or otherwise treated

before use, the specimens for tensile test representing such material shall be cut from properly annealed or similarly treated short lengths of the full section of the bar.

9. At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing $\frac{3}{8}$ in. and more in thickness is rolled from one melt a test shall be made from the thickest and thinnest material rolled.

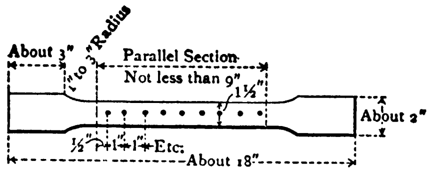


FIG. 1.

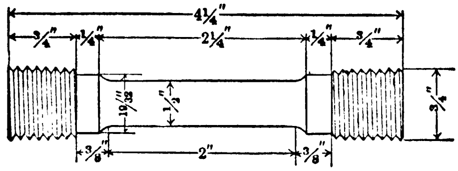


FIG. 2.

10. For material less than $\frac{5}{16}$ in. and more than $\frac{3}{4}$ in. in thickness, the following modifications will be allowed in the requirements for elongation:

(*a*) For each $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in., a deduction of $2\frac{1}{2}$ from the specified percentage will be allowed.

(*b*) For each $\frac{1}{8}$ in. in thickness above $\frac{3}{4}$ in., a deduction of 1 from the specified percentage will be allowed.

11. Bending tests may be made by pressure or by blows. Plates, shapes, and bars less than 1 in. thick shall bend as called for in Section 2.

12. Angles $\frac{3}{4}$ in. and less in thickness shall open flat, and angles $\frac{1}{2}$ in. and less in thickness shall bend shut, cold, under blows of a hammer, without sign of fracture. This test will be made only when required by the inspector.

13. Rivet steel, when nicked and bent around a bar of the same diameter as the rivet rod, shall give a gradual break and a fine, silky, uniform fracture.

14. Finished material shall be free from injurious seams, flaws, cracks, defective edges, or other defects, and have a smooth, uniform, workmanlike finish. Plates 36 in. in width and less shall have rolled edges.

15. Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it. Steel for pins shall be stamped on the end. Rivet and lattice steel and other small parts may be bundled, with the above marks on an attached metal tag.

16. Material which, subsequent to the foregoing tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks, or other imperfections, or is found to have injurious defects, will be rejected at the shop, and shall be replaced by the manufacturer at his own cost.

17. A variation in cross-section or weight of each piece of steel of more than $2\frac{1}{2}$ per cent from that specified will be sufficient cause for rejection, except in cases of sheared plates, which will be covered by the following permissible variations, which are to apply to single plates:

Plates weighing $12\frac{1}{2}$ lb. per sq. ft. or more:

(*a*) Up to 100 in. wide, $2\frac{1}{2}$ per cent above or below the prescribed weight;

(*b*) 100 in. wide or more, 5 per cent above or below.

Plates weighing less than $12\frac{1}{2}$ lb. per sq. ft.:

(*a*) Up to 75 in. wide, $2\frac{1}{2}$ per cent above or below;

(*b*) 75 in., and up to 100 in. wide, 5 per cent above or 3 per cent below;

(*c*) 100 in. wide or more, 10 per cent above or 3 per cent below.

18. Plates will be accepted if their thickness is not more than 0.01 in. less than that ordered.

19. An excess over the nominal weight, corresponding to the dimensions on the order, will be allowed for each plate, if not more than that shown in Table IV, 1 cu. in. of rolled steel being assumed to weigh 0.2833 lb.

Cast Iron.—20. Except where chilled iron is specified, castings shall be made of tough, gray iron, with not more than 0.10 per cent of sulphur. They shall be true to patterns, out of wind, and free from flaws and excessive shrinkage. If tests are demanded, they shall be made on the

TABLE IV.

Thickness, in Inches.	Nominal Weight in Pounds per Square Foot.	Width of Plates.		
		Up to 75 In.	75 In. and up to 100 In.	100 In. and up to 115 In.
$\frac{1}{4}$	10.20	10 per cent	14 per cent	18 per cent
$\frac{5}{16}$	12.75	8 " "	12 " "	16 " "
$\frac{3}{8}$	15.3	7 " "	10 " "	13 " "
$\frac{7}{16}$	17.85	6 " "	8 " "	10 " "
$\frac{1}{2}$	20.4	5 " "	7 " "	9 " "
$\frac{9}{16}$	22.95	$4\frac{1}{2}$ " "	$6\frac{1}{2}$ " "	$8\frac{1}{2}$ " "
$\frac{5}{8}$	25.5	4 " "	6 " "	8 " "
More than $\frac{5}{8}$	....	$3\frac{1}{2}$ " "	5 " "	$6\frac{1}{3}$ " "

"Arbitration Bar" of the American Society for Testing Materials, which is round bar, $1\frac{1}{4}$ in. in diameter and 15 in. long. The transverse test shall be made on a supported length of 12 in. with the load at the middle. The minimum breaking load thus applied shall be 2,900 lb., with a deflection of at least $\frac{3}{16}$ in. before rupture.

Workmanship, Inspection, and Painting.—21. All parts forming the structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best in modern shop practice.

22. All material shall be thoroughly straightened in the shop, by methods which will not injure it, before being laid off or worked in any way.

23. The shearing shall be done neatly and accurately, and all portions of the work exposed to view shall have a neat and uniform appearance.

24. The size of each rivet, called for by the plans, shall be understood to mean the actual size of the cold rivet before it is heated.

25. All plates and shapes shall be shaped to the proper curve by cold rolling; heating or hammering for straightening or curving will not be allowed.

26. Plates to be scarfed may be heated to a cherry-red color, but not hot enough to ignite a piece of dry wood when applied to it. Most careful attention shall be paid to all scarfing.

27. All plates or shapes shall be punched before being bevel-sheared or planed for caulking.

28. All screw threads shall make tight fits in the nuts and turnbuckles, and shall be United States Standard, except for diameters greater than $1\frac{3}{8}$ in., when they shall have six threads per inch. The dimensions of screws of various sizes shall be as follows:

Diameter of screw ends...............................	1 in.	$1\frac{1}{8}$ in.	$1\frac{1}{4}$ in.	$1\frac{3}{8}$ and greater
Number of threads per inch...........................	8	7	7	6

The minimum excess at the root of the thread over the body of the bar shall be 15 per cent. The shape of the thread shall be U. S. Standard.

TABLE V.

Standard Upsets for Round and Square Bars.

Round Bars.		Square Bars.	
Bar.	Upset.	Bar.	Upset.
Diameter, in Inches.	Diameter, in Inches.	Side, in Inches.	Diameter, in Inches.
$\frac{3}{4}$	1	$\frac{3}{4}$	$1\frac{1}{8}$
$\frac{7}{8}$	$1\frac{1}{8}$	$\frac{7}{8}$	$1\frac{1}{4}$
1	$1\frac{3}{8}$	1	$1\frac{1}{2}$
$1\frac{1}{8}$	$1\frac{1}{2}$	$1\frac{1}{8}$	$1\frac{5}{8}$
$1\frac{1}{4}$	$1\frac{5}{8}$	$1\frac{1}{4}$	$1\frac{7}{8}$
$1\frac{3}{8}$	$1\frac{3}{4}$	$1\frac{3}{8}$	2
$1\frac{1}{2}$	$1\frac{7}{8}$	$1\frac{1}{2}$	$2\frac{1}{4}$
$1\frac{5}{8}$	2	$1\frac{5}{8}$	$2\frac{3}{8}$
$1\frac{3}{4}$	$2\frac{1}{8}$	$1\frac{3}{4}$	$2\frac{1}{2}$
$1\frac{7}{8}$	$2\frac{1}{4}$	$1\frac{7}{8}$	$2\frac{3}{4}$
2	$2\frac{3}{8}$	2	$2\frac{7}{8}$

29. The diameter of the die used in punching rivet holes shall not exceed that of the punch by more than $\frac{1}{16}$ in. All rivet holes shall be punched, except as stated in Part I, Section 19.

30. All punched and reamed bolts shall be clean cuts, without torn or ragged edges. The burrs on all reamed holes shall be removed by a tool, countersinking not more than $\frac{1}{16}$ in. Any parts of the structure in which difficulties may arise in field riveting, shall be assembled in the shop and marked properly before shipment.

31. Rivet holes shall be accurately spaced; eccentrically located rivet holes, if not sufficient to cause rejection shall be corrected by reaming, and rivets of larger size shall be used in the holes thus reamed.

32. The use of drift-pins will be allowed only for bringing together several parts forming part of the structure; force will not be allowed to be used in drifting under any circumstances.

33. The use of sledges in driving or hammering any part of the structure will not be allowed. Care shall be taken to prevent material from falling, or from being in any way subjected to heavy shocks.

34. Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand-driving. All rivet heads shall be concentric with the holes.

35. All caulking shall be done with a round-nosed tool, and only by experienced and skilled men. Caulking around rivet heads will not be allowed. All leaky rivets shall be cut out and replaced with new ones. All fractured material shall be replaced free of cost to the owner.

36. If the owner furnishes an inspector, he shall have full access, at all times to all parts of the shop where material under his inspection is being manufactured.

37. The inspector shall stamp with a private mark each piece accepted. Any piece not thus marked may be rejected at any time, and at any stage of the work. If the inspector, through oversight or otherwise, has accepted material or work which is defective or contrary to these specifications, this material, no matter in what stage of completion, may be rejected by the owner.

Painting and Testing.—38. Before leaving the shop, all steel work excepting the laps in contact on the tank work, shall receive one coat of approved paint or boiled linseed oil. All parts which will be inaccessible after erection shall be well painted, except as stated before.

39. After the structure is erected and all seams have been caulked, it shall be tested for water-tightness, and leaky places shall be caulked or marked. The water shall then be discharged and the leaky seams shall be caulked. Leaky rivets shall be treated as per Section 35. After the structure has been standing empty for 3 days it shall be retested, and then, if all joints are water-tight, it shall be given one coat of approved paint both inside and outside of the tank or stand-pipe. Painting in the open air shall never be done in wet or freezing weather. The owner will select the color of the final coat of paint.

40. The contractor shall guarantee the tightness of the tank, or stand-pipe, against leakage, when filled with the liquid it is designed to contain.

PART IV. FOUNDATIONS FOR ELEVATED TANKS ON TOWERS, AND FOR STAND-PIPES.

1. The average permissible pressure on the soil is as follows:

Soft clay	1 ton per sq. ft.
Ordinary clay	2 tons per sq. ft.
Dry sand and dry clay	3 tons per sq. ft.
Hard clay	4 tons per sq. ft.
Gravel and coarse sand	6 tons per sq. ft.

2. In all cases a thorough investigation of the ground and the site shall be made before proceeding with the foundations.

3. All foundations shall be carried below the frost line, and the anchor-bolts shall be placed deep enough to develop their full strength.

4. In foundations for towers with inclined legs supporting elevated tanks care shall be taken that the piers are constructed in such a manner, that the resultant of the vertical and horizontal forces, due to direct loads, passes through the center of gravity of the piers.

5. Foundations, in general, shall be of concrete composed of 1 part Portland cement, 3 parts sand, and 5 parts crushed stone or gravel. In special cases, where part of the foundation is under water, the concrete shall be a 1 : 2 : 4 mixture.

Note.—For specifications for mixing and placing the concrete in the foundations, see Chapter V.

GENERAL SPECIFICATIONS FOR STEEL WATER AND OIL TANKS.*

1. **Scope of Specifications.**—These specifications are intended for steel tanks requiring plates not more than $\frac{5}{8}$ in. thick.

2. **Quality of Metal.**—The metal in these tanks shall be open-hearth steel. The steel shall conform in physical and chemical properties to the specifications of this Association for steel bridges.

3. **Loading.**—The weight of water shall be assumed to be 63 lb., crude oil 56 lb., and creosote oil 66 lb. per cu. ft. Wind pressure, acting in any direction, shall be assumed to be, in pounds, 30 times the product of the height by two-thirds of the diameter of the tank in feet.

4. **Unit Stresses.**—Unit stresses shall not exceed the following:

(*a*) Tension in plates, 15,000 lb. per sq. in. on net section.

(*b*) Shear in plates, 12,000 lb. per sq. in. on net section.

(*c*) Shear on rivets, 12,000 lb. per sq. in. on net section.

(*d*) Bearing pressure on field rivets, 20,000 lb. per sq. in.

5. **Cylindrical Rings.**—Plates forming the shell of the tank shall be cylindrical and of different diameters, in and out, from course to course.

6. **Workmanship.**—All workmanship shall be first-class. All plates shall be beveled on all edges for caulking after being punched. The punching shall be from the surface to be in contact The plates shall be formed cold to exact form after punching and beveling. All rivet holes shall be accurately spaced. Drift pins shall be used only for bringing the parts together. They shall not be driven with enough force to deform the metal about the holes. Power riveting and caulking should be used. A heavy yoke or pneumatic bucker shall be used for power driven rivets. Riveting shall draw the joints to full and tight bearing.

7. **Caulking.**—The tank shall be made water or oil tight by caulking only. No foreign substance shall be used in the joints. For water tanks, the caulking shall preferably be done on the inside of tank and joint only; but for oil tanks the caulking should be done on both sides. No form of caulking tool or work that injures the abutting plate shall be used.

8. **Minimum Thickness of Plates.**—The minimum thickness of plates in the cylindrical part of the tank shall not be less than $\frac{1}{4}$ in. and in flat bottoms not less than $\frac{5}{16}$ in. In curved bottoms the thickness of plate shall be not less than that of the lower plate in the cylindrical part.

9. **Horizontal and Radial Joints.**—Lap joints shall generally be used for horizontal seams and splices and for radial seams in curved bottoms.

10. **Vertical Joints.**—For vertical seams and splices, lap joints shall be used with plates not more than $\frac{3}{8}$ in. thick. With thicker plates, double butt joints with inside and outside straps shall generally be used. The edge of the plate in contact at the intersection of horizontal and vertical lap joints shall be drawn out to a uniform taper and thin edge.

11. **Rivets, Rivet Holes, Punching and Pitch.**—For plates not more than $\frac{3}{8}$ in. thick, $\frac{5}{8}$ in. rivets shall be used. For thicker plates, $\frac{3}{4}$ in. rivets shall be used. The diameter of rivet holes shall be $\frac{1}{16}$ in. larger than the diameter of the rivets used. The punching shall conform to the specifications of this Association for such work on steel bridges. A close pitch, with due regard for thickness of plate and balanced stress between tension on plates and shear on rivets, is desirable for caulking.

12. **Tank Support.**—If the tank is supported on a steel substructure, the latter shall conform to the specifications of this Association for the manufacture and erection of steel bridges, except that allowance shall be made for wind pressure, but not for impact.

13. **Painting.**—In the shop the metal shall be cleaned of dirt, rust and scale and, except the surfaces to be in contact in the joints of the tank, shall be given a shop coat of paint or metal preservative selected and applied as specified by the company.

After being completely erected, caulked and cleaned of dirt, rust and scale, all exposed metal work shall be painted or treated with such coat or coats of paint or metal preservative as shall be selected by the railway company.

14. **Plans and Specifications.**—Under these specifications and in conformity thereto the railway company shall cause to be prepared or shall approve detailed plans and specifications for such tanks, herein specified, as it shall construct. Such plans and specifications shall cover all necessary tank auxiliaries.

REFERENCES. Hazlehurst's "Towers and Tanks for Waterworks," second edition, 1904, published by John Wiley & Sons, covers the design and construction of steel stand-pipes and steel elevated tanks on steel towers, and supplements the data and discussion in this chapter. Considerable data on the design and construction of stand-pipes and elevated tanks on towers for railway service are given in the annual reports of the proceedings of the American Railway Engineering Association, particular reference is made to volume 11, part 2; volume 12, part 3, and volume 13.

* Adopted, Am. Ry. Eng. Assoc., Vol. 13, 1912.

CHAPTER XIa.

Self-Supporting Steel Stacks.

Introduction.—Self-supporting steel stacks are usually made with the upper part cylindrical and with the lower part flared. The height of the flared or bell mouth base depends upon the breaching, the location of the flue opening and upon the required diameter of the base of the stack. The height of the flare will vary from one-eighth to one-fourth the height. Where the height of the flare is one-fourth the height of the stack, and the top of the cone is at the top of the stack, the diameter of the base with a conical flare will be one-third greater than the diameter of the upper part of the stack. The ratio of the diameter of the base of the flare to the diameter of the stack in well designed stacks varies from $\frac{4}{3}$ to $\frac{5}{4}$. The bell mouth stack is more expensive to build than a flared stack, and has no advantage.

The plates in the stack shell vary from 4 to 7 ft. in height, 5 ft. being the most common height of course. Lap joints are used for vertical seams and for horizontal seams in the upper part of the stack. Butt joints are used for horizontal seams with heavy plates, and in the flare of stacks. With lap joints the rings are made conical with each ring telescoping over the ring below. The edges of the plates should be beveled for caulking. The riveted joints should be designed for caulking, and the joints should all be caulked after erection.

Self-supporting stacks are made with lining and also are left unlined. While many unlined steel stacks have given excellent service, it is now the general practice to line all self-supporting steel stacks. The lining may be made of radial fire brick, common brick, concrete, cement gunite, or a material such as vitrobestos. The lining should be carried the full height of the stack. The lining in the upper part of the stack is usually supported on steel angles riveted to the inside of the stack. These angles are spaced vertically from 5 ft. to 15 ft. apart. When lined with brick, the lower section of the stack is usually lined with fire brick laid in fire clay, while the upper section is lined with common brick laid in cement mortar. For an independent lining an 8-in. brick wall will be required for the lower half, and a 4-in. brick wall for the upper half of the stack. The space between the lining and the steel shell should be filled with cement mortar to prevent condensation of moisture inside the shell with resulting corrosion. Cement gunite linings about 2 in. thick are now being used. The reinforcing fabric is fastened to horizontal angles and the cement mortar is applied with the cement gun. While gunite lining has been in use for only a short time, it would appear to make a very satisfactory lining.

To retard corrosion the steel plates are painted one coat of paint in the shop and usually two coats of heat resisting paint after erection. A graphite or carbon paint or other tried heat resisting paint should be used. The plates in the steel shell should be made at least one-sixteenth inch thicker than required by the stresses to provide for corrosion. No plates thinner than $\frac{1}{4}$ in. should be used for stack plates. Steel having an admixture of from 0.25 to 0.30 per cent copper is more resistant to corrosion than is steel or iron not containing copper. Copper bearing steel should be used for the steel plates in self-supporting steel stacks. The so called "ingot irons" or "pure irons" have no advantage over structural steel for use in steel stacks.

To prevent the collapse of the upper part of the stack the top of the stack should be reinforced. The reinforcement is commonly made by riveting a painter's trolley track on the outside near the top. The horizontal angles on the inside to carry the lining also stiffen the stack against collapse due to wind pressure. A cleanout door should be provided near the bottom, unless it is possible to clean out the stack through the foundation. Ladders are provided on the outside and sometimes on both outside and inside. For high stacks safety rings should be fastened to the ladder as shown in Fig. 5 and Fig. 6.

In erection the plates should be drawn up tight before riveting. Not less than one-third of the holes should be filled with field bolts, well distributed in the joint and drawn up tight before driving rivets.

The foundations of self-supporting steel stacks should be made massive in order that the vibrations due to wind may be made as small as possible. Where self-supporting steel stacks are carried on a structural steel framework, the supporting girders and columns should be encased in concrete to provide a mass to reduce vibrations due to wind.

Data for Design.—The following data will be of assistance in the design of self-supporting steel stacks:

Notation:

h = distance in feet of any point below the top of the stack.
d = diameter of the stack in feet.
r = radius of the stack in feet.
r_1 = outside radius of the steel shell in feet.
r_2 = inside radius of the steel shell in feet.
t = thickness of steel shell in inches.
p = pressure of the wind on a projected diameter in pounds per square foot.
P = total wind pressure on the stack in pounds.
W_s = weight of steel shell above any point in pounds.
W_l = weight of stack lining above any point in pounds.
W_f = weight of foundation in pounds.
b = diameter of foundation (assuming a circular section) in feet.
h_1 = height of foundation in feet.
d_1 = diameter of anchor bolt circle in feet.
g = spacing of anchor bolts in inches.
M = bending moment due to wind in inch-pounds.
F = stress per lineal inch along a circumferential joint.
f = stress along a circumferential joint in pounds per square inch.
S = stress per vertical lineal inch of stack in pounds.
S' = stress in vertical section of stack in pounds per square inch.
T = total stress in an anchor bolt in pounds.
a = thickness of steel base plate in inches.
m = width of steel base plate in inches.
c = projection of steel base plate in inches.
f_c = allowable pressure of base plate on masonry in lb. per sq. in.

Wind Pressure.—The pressure of the wind on a structure depends on the shape of the structure, the height of the structure, the width of the structure, on the location, and on climatic conditions. Experiments have shown that the wind pressure per sq. ft. increases with the height of the structure. Professor Henry Adams in "Mechanics of Building Construction" gives the following formula for wind pressure on a surface:

$$\log p = 1.125 + 0.32 \log h_1 - 0.12 \log w$$

where p = wind pressure in lb. per sq. ft., h_1 = height of center of gravity of surface above ground level in ft.; w = width of surface in feet. For a stack 20 ft. in diameter the wind pressure from the above formula will be as follows:—Height = h = 40 ft., p = 24.3 lb.; h = 100 ft., p = 32.5 lb.; h = 200 ft., p = 40.7 lb.; h = 300 ft., p = 57.7 lb. The height of the stack h will equal $2h_1$ in the formula above.

German specifications for design of tall chimneys specify wind loads per sq. ft. as follows: 26 lb. for rectangular chimneys; 67 per cent of 26 lb. on circular chimneys; and 71 per cent of 26 lb. on octagonal chimneys.

A wind load of 33⅓ lb. per sq. ft. on the projection of the chimney was used in the design of the brick chimney built for the Boston and Montana Company, Butte, Montana. The chimney is 76 ft. in diameter at the base and 50 ft. in diameter at the top, and is 506 ft. in height.

The wind pressure on the steel stack 400 ft. high and 30 ft. in diameter built by the United Verdi Copper Company at Jerome, Arizona, was taken as 25 lb. per sq. ft. of vertical projection of the stack.

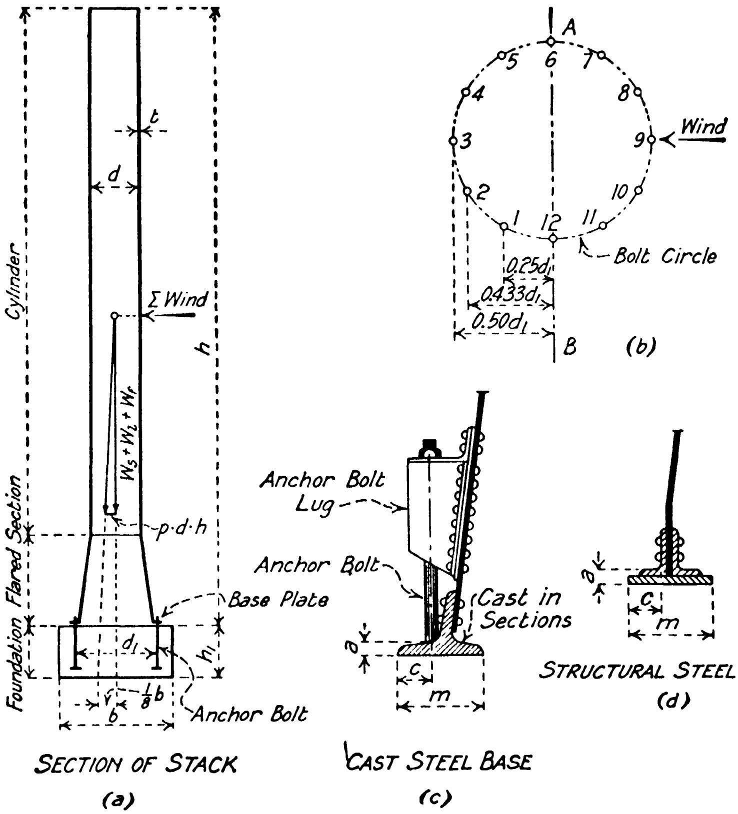


FIG. 1.

The maximum wind pressure on a square stack is commonly taken as from 30 to 40 lb. per sq. ft. of the projection normal to the wind. The wind pressure on a circular stack is taken as two-thirds of the pressure on a square stack, and the pressure will vary from 20 to 25 lb. per sq. ft. acting normal to the projection of the stack. A pressure of 20 lb. per sq. ft. is adequate for a small stack in a protected location, while 25 lb. per sq. ft. should be used for a large stack or for a stack in an exposed location.

DESIGN OF STEEL STACK.—The total lateral pressure P, acting above any section at a distance h below the top of the stack, (a), Fig. 1, will be

$$P = p \cdot d \cdot h \tag{1}$$

and the bending moment in ft.-lb. at any distance h below the top of the stack will be

$$M = \tfrac{1}{2} p \cdot d \cdot h^2 \tag{2}$$

The wind pressure will be taken at $37\frac{1}{2}$ lb. per sq. ft. on two-thirds of the projected area or 25 lb. per sq. ft. on the entire projected area. Substituting in (2), and reducing to in.-lb.

$$M = (25d \cdot h^2 \cdot 12)/2$$
$$= 150d \cdot h^2 \tag{3}$$

The unit stress in the extreme fiber of the shell is

$$f = M \cdot c/I \tag{4}$$

Now $c = 12r_1$, $I = \pi(r_1^4 - r_2^4)/4 = \pi \cdot r^3 \cdot t$

(approx.; r_1 is in ft., and t is in in.) $= 12^3\pi \cdot r^3 \cdot t = 1{,}728\pi \cdot r^3 \cdot t$

Substituting M from (2) and c, and I as above in (4)

$$f = \frac{6p \cdot d \cdot h^2 \times 12r}{1{,}728\pi \cdot r^3 \cdot t}$$
$$= \frac{0.0533p \cdot h^2}{t \cdot d} \tag{5}$$

For $p = 25$ lb. per sq. ft.

$$f = \frac{1.33h^2}{t \cdot d} \tag{5'}$$

The stress per lineal inch along the circumference is

$$F = \frac{0.0533p \cdot h^2}{d} \tag{6}$$

For $p = 25$ lb. per sq ft.

$$F = \frac{1.33h^2}{d} \tag{6'}$$

The stress per lineal inch on any circumference is also equal to the moment M divided by the area of the stack

$$F = \frac{6p \cdot d \cdot h^2}{\pi \cdot r^2 \times 12 \times 12}$$
$$F = \frac{0.0533p \cdot h^2}{d} \tag{6}$$

If f_t = allowable unit stress on the net section, and e = efficiency of the joint, from (5) the thickness of the plate will be

$$t = \frac{0.0533p \cdot h^2}{d \cdot f_t \cdot e} \tag{7}$$

For $p = 25$ lb. per sq. ft.

$$t = \frac{1.33h^2}{d \cdot f_t \cdot e} \tag{7'}$$

The unit stress due to the weight of the stack above the section is

$$f_s = \frac{W_s}{12\pi \cdot d \cdot t} \tag{8}$$

Assuming that the stack plates are of constant thickness above the horizontal joint

$$f_s = \frac{490\pi \cdot d \cdot h \cdot t}{144\pi \cdot d \cdot t}$$
$$= 3.4h \text{ (approx.)} \tag{8'}$$

From equation (8′) it will be seen that if $h = 300$ ft., the stress due to weight will be 1,020 lb. per sq. in. For stacks of height less than about 200 ft., the weight of the steel stack may be omitted (see § 6, specifications at end of chapter).

For steel stacks with lining the load on the horizontal joints should be calculated. For a fire brick lining 4 in. thick, the l ad per lineal inch on a joint, as uming that brick weighs 125 lb. per cu. ft., will be

$$F_1 = \frac{W_l}{1 \quad \pi \cdot d} \tag{9}$$

$$= \frac{4 \times 125\pi \cdot d \cdot h}{144\pi \cdot d}$$

$$= 3.48h \tag{9′}$$

The weight of the stack and the weight of the lining should be omitted in calculating the maximum tension, for the reason that the maximum wind load may come on the stack before the stack is lined. The weight of the stack and the lining should be included in calculating the compressive stresses, see § 6 specifications at end of chapter.

Stresses in Anchor Bolts.—The stresses in the anchor bolts will be calculated on the assumption that all anchor bolts are drawn up tight, and that rotation will be about a line through the center of gravity of the base of the stack and at right angles to the direction of the wind, see (b) Fig. 1.

From equation (6) the stress per lineal inch of shell for a stack of uniform diameter is

$$F = \frac{0.0533p \cdot h^2}{d} \tag{6}$$

For $p = 25$ lb. per sq. ft.

$$F = \frac{1.33h^2}{d} \tag{6′}$$

For a stack with a bell bottom if d is diameter of stack and d_1 is diameter of base of bell

$$F = \frac{0.0533p \cdot h^2 \cdot d}{d_1^2} \tag{10}$$

For $p = 25$ lb. per sq. ft.

$$F = \frac{1.33h^2 \cdot d}{d_1^2} \tag{11}$$

If g is the spacing of the anchor bolts in inches, when the diameter of the anchor bolt circle is equal to diameter of stack, the stress in an anchor bolt due to wind will be

$$T = \frac{1.33g \cdot h^2}{d} \tag{12}$$

Where the diameter of the anchor bolt circle is d_1

$$T = \frac{1.33g \cdot h^2 \cdot d}{d_1^2} \tag{12′}$$

The tension in the anchor bolt due to wind will be reduced by the compressive stress due to the weight of the steel stack (the maximum win l load may occur during erection and the weight of the lining may be neglected) and

$$T_1 = \frac{1.33g \cdot h^2 \cdot d}{d_1^2} - \frac{g \cdot W_s}{12\pi \cdot d} \tag{12″}$$

If the anchor bolt is given an initial stress, this stress should be added to (12″).

Design of Base Plate.—The compression per lineal inch due to wind, from equation (10) will be

$$F = \frac{0.0533p \cdot h^2 \cdot d}{d_1^2} \tag{10}$$

The pressure due to the weight of the steel, W_s, and lining W_l will be

$$F' = \frac{W_s + W_l}{12\pi \cdot d_1} = \frac{W_s + W_l}{37.7d_1} \tag{13}$$

The maximum compression per lineal inch on the base plate will be

$$F_1 = \frac{0.0533p \cdot h^2 \cdot d}{d_1^2} + \frac{W_s + W_l}{37.7d_1} \tag{14}$$

Width of Steel Base Plate.—Let f_c = allowable pressure of the steel base plate on the masonry in lb. per sq. in.; f_t = allowable tensile stress in the steel base plate in lb. per sq. in.; m = width of base plate in inches, c = projection of base plate in inches, and a = thickness of base plate, (c) Fig. 1. The base plate will be a cantilever beam with thickness a and span c. From mechanics the bending moment per inch of length will be

$$M = \frac{f_c \cdot c^2}{2} = \frac{f_t \cdot I}{a/2} = \frac{f_t \cdot a^2}{6}$$

and solving

$$a = c\sqrt{\frac{3f_c}{f_t}} \tag{15}$$

The width of base plate will be

$$m = F_1/f_c \tag{16}$$

where F_1 is the maximum compression on the base plate in lb. per lineal inch as calculated by formula (14).

Thickness of Plates in Steel Stacks.—The thickness of plates in self-supporting steel stacks of different diameters and heights, calculated from formula (7′) for a joint efficiency, e = 50 per cent is given in Table I, and for e = 70 per cent is given in Table II. In designing a steel stack the preliminary thickness of plates may be taken from these tables. The preliminary design should be checked by investigating for weight of steel using formula (8), and for weight of lining using formula (9). For the thickness of plates in self-supporting steel stacks designed by Babcock and Wilcox Co., see Table V.

Design of Foundations.—The overturning moment at the top of the foundation is

$$M = \tfrac{1}{2}p \cdot d \cdot h^2 \tag{2}$$

The resisting moment will be calculated on the assumption that for average conditions

$$h_1 = 0.4b \tag{17}$$

where h_1 is height of footing and b is the diameter of the foundation. The volume of the footing in cu. ft. will be

$$V = 0.7854b^2 \cdot h_1$$

$$\text{Weight of Foundation} = W_f = 0.7854b^2 \times 0.4b \times 150 = 47.125b^3$$

Now the overturning moment about the base of the foundation in ft.-lb. will be

$$M = p \cdot d \cdot h(\tfrac{1}{2}h + h_1) \tag{18}$$

If the resultant thrust due to wind pressure and weight of steel stack, lining and foundation does not fall outside the middle quarter of the foundation, which is the usual condition, then

$$M = (W_s + W_l + W_f)\, b/8$$

TABLE I

Thickness of Plates for Self-Supporting Steel Stacks.

Wind load = 25 lb. per sq. ft. Efficiency of joints e = 0.50.
Allowable tensile stress f_t = 12,000 lb. sq. in. Thickness in inches.

Height ft.	Diameter of Stack in ft.								
	4	5	6	7	8	9	10	11	12
60	.199								
70	.271	.217							
80	.354	.283	.236	.202					
90	.448	.358	.298	.256	.224	.199			
100	.553	.442	.368	.316	.276	.246	.221	.200	
110	.669	.535	.446	.382	.334	.297	.267	.243	.223
120	.796	.637	.530	.455	.398	.354	.318	.289	.265
130		.747	.623	.534	.467	.415	.374	.340	.311
140			.722	.619	.542	.481	.433	.394	.361
150				.711	.622	.553	.497	.452	.414
160					.707	.629	.566	.514	.472
170						.710	.638	.581	.532
180							.716	.651	.597
190								.725	.665
200									.737

Height ft.	Diameter of Stack in ft.								
	14	16	18	20	22	24	26	28	30
150	.355	.311	.276	.249					
160	.404	.354	.314	.283	.257				
170	.456	.399	.355	.319	.290	.266	.246		
180	.512	.448	.398	.358	.326	.298	.275	.256	
190	.570	.499	.443	.399	.363	.332	.307	.285	.266
200	.632	.553	.491	.442	.402	.368	.340	.316	.295
220	.764	.669	.594	.535	.488	.446	.411	.382	.357
240		.796	.707	.636	.579	.530	.490	.455	.424
260				.747	.679	.623	.575	.534	.498
280					.788	.722	.667	.619	.577
300							.765	.711	.663

and

$$b = \frac{8M}{W_s + W_l + W_f} = \frac{8p\cdot d\cdot h\,(\frac{1}{2}h + h_1)}{W_s + W_l + W_f} \tag{18'}$$

Approximate Formula for Foundation.—If the resultant thrust due to wind pressure does not fall outside the middle quarter of the circular base (the kern of the base is a circle with a diameter $\frac{1}{4}b$), and if the weight of the steel stack be neglected, the resisting moment will be

$$M^1 = W_f\cdot b/8 \tag{19}$$

equating (18) and (19), and solving for b, the height of the foundation

$$b^4 = \frac{8p\cdot d\cdot h\,(\frac{1}{2}h+h_1)}{47.125}.$$

neglecting h_1 and solving

$$b = 0.54p^{1/4}\cdot d^{1/4}\cdot h^{1/2} \quad \text{(approx.)} \tag{20}$$

For $p = 25$ lb. per sq. ft.

$$b = 1.207d^{1/4}\cdot h^{1/2} \quad \text{(approx.)} \tag{20'}$$

Equation (20) is an approximate formula for the diameter of a circular foundation in terms of the diameter of the stack, d, and the height of the stack, h, all dimensions in feet. The wind pressure on the added surface due the bell and the foundation and the weight of the stack and lining are neglected.

Pressure on Foundations.—The maximum pressure on the leeward edge of the foundation will be equal to

$$p_1 = \frac{W_s + W_l + W_f}{0.7854b^2} + \frac{M\cdot b}{2I}$$

$$= \frac{W_s + W_l + W_f}{0.7854b^2} + \frac{32M}{\pi b^3} \tag{21}$$

TABLE II.

THICKNESS OF PLATES FOR SELF-SUPPORTING STEEL STACKS.

Wind load = 25 lb. per sq. ft. Efficiency of joints $e = 0.70$.
Allowable tensile stress, $f_t = 12{,}000$ lb. sq. in. Thickness in inches.

Height ft.	Diameter of Stack in ft.								
	4	5	6	7	8	9	10	11	12
70	.193								
80	.253	.202							
90	.320	.256	.213	.183					
100	.395	.316	.263	.226	.197				
110	.478	.382	.318	.273	.239	.212	.191		
120	.568	.455	.379	.325	.284	.253	.227	.207	
130	.667	.534	.445	.381	.334	.296	.267	.243	.222
140	.774	.619	.516	.442	.387	.344	.309	.281	.258
150		.711	.592	.508	.444	.395	.355	.323	.296
160			.674	.577	.505	.449	.404	.367	.337
170				.652	.570	.507	.456	.415	.380
180					.639	.568	.512	.465	.426
190					.713	.633	.570	.518	.475
200						.702	.632	.574	.526
220							.764	.695	.637

Height ft.	Diameter of Stack in ft.								
	14	16	18	20	22	24	26	28	30
160	.289	.253	.225						
170	.326	.285	.254	.228					
180	.365	.320	.284	.256	.233				
190	.407	.356	.317	.285	.259	.238			
200	.451	.395	.351	.316	.287	.263	.243		
220	.546	.478	.425	.382	.347	.318	.294	.273	.255
240	.650	.568	.505	.455	.413	.379	.350	.325	.303
260		.667	.593	.534	.485	.445	.411	.381	.356
280			.688	.619	.563	.516	.476	.442	.413
300				.710	.646	.592	.547	.508	.474
320						.674	.622	.577	.539

Where the resultant thrust is kept within the kern of the section (middle quarter), the maximum pressure on the leeward side will be

$$p_1 = \frac{2(W_s + W_l + W_f)}{0.7854b^2} \tag{22}$$

The maximum pressure should be kept low, see specifications for elevated steel tanks, Chapter XI. If there is danger of settlement the foundation should be carried on piles.

The diameters of circular concrete foundations for self-supporting steel stacks as calculated by equation (20′) are given in Table III. The depth of foundation is four-tenths the diameter of the foundation in each case.

TABLE III.

DIAMETER OF CIRCULAR MASONRY FOUNDATIONS FOR SELF-SUPPORTING STEEL STACKS.

Height of Foundation = ⅖ its diameter. Diameter of Foundations in feet.

p = 25 lb. per sq. ft.

Height ft.	Diameter of Steel Stack in ft.								
	4	5	6	7	8	9	10	11	12
70	14.3	15.1	15.8	16.4	17.0	17.5	18.0	18.4	18.8
80	15.3	16.1	16.9	17.6	18.2	18.7	19.2	19.7	20.1
90	16.2	17.1	17.9	18.6	19.3	19.8	20.4	20.9	21.3
100	17.1	18.0	18.9	19.6	20.3	20.9	21.5	22.0	22.5
110	17.9	18.9	19.8	20.6	21.3	21.9	22.5	23.1	23.6
120	18.7	19.8	20.7	21.5	22.2	22.9	23.5	24.1	24.6
130	19.5	20.6	21.5	22.4	23.1	23.8	24.5	25.1	25.6
140	20.2	21.4	22.4	23.2	24.0	24.7	25.4	26.0	26.6
150	20.9	22.1	23.1	24.1	24.9	25.6	26.3	26.9	27.5
160		22.8	23.9	24.8	25.7	26.5	27.2	27.8	28.4
170			24.6	25.6	26.5	27.3	28.0	28.7	29.3
180				26.4	27.2	28.1	28.8	29.5	30.1
190					28.0	28.8	29.6	30.3	31.0
200					28.7	29.6	30.4	31.1	31.8
220					30.1	31.0	31.8	32.6	33.3
240						32.4	33.2	34.1	34.8

Height ft.	Diameter of Steel Stack in ft.								
	14	16	18	20	22	24	26	28	30
120	25.6	26.4	27.3	28.0	28.7	29.3	29.8	30.4	30.9
130	26.6	27.5	28.4	29.1	29.8	30.5	31.1	31.7	32.2
140	27.6	28.6	29.4	30.2	30.9	31.6	32.2	32.9	33.4
150	28.6	29.6	30.5	31.3	32.0	32.7	33.4	34.0	34.6
160	29.5	30.5	31.5	32.3	33.1	33.8	34.5	35.1	35.7
170	30.5	31.5	32.4	33.3	34.1	34.8	35.5	36.2	36.8
180	31.3	32.4	33.4	34.3	35.1	35.9	36.6	37.3	37.9
190	32.2	33.3	34.3	35.2	36.0	36.8	37.6	38.3	38.9
200	33.0	34.1	35.2	36.1	37.0	37.8	38.5	39.3	39.9
220	34.6	35.8	36.9	37.9	38.8	39.6	40.4	41.2	41.9
240	36.2	37.4	38.5	39.5	40.5	41.4	42.2	43.0	43.8
260	37.6	38.9	40.1	41.2	41.2	43.1	43.9	47.8	45.6
280	39.1	40.4	41.6	42.7	43.7	44.7	45.6	46.5	47.3
300	40.4	41.8	43.1	44.2	45.3	46.3	47.2	48.1	48.9
320	41.8	43.2	44.5	45.7	46.7	47.8	48.8	49.7	50.5

ALLOWABLE STRESSES.—The self-supporting steel stack for the United Verde Company, Jerome, Ariz., 400 ft. high by 30 ft. in diameter, was designed for a wind pressure of 25 lb. per sq. ft. on the vertical projection of the stack with the following working stresses: Tension 16,000 lb. per sq. in. on net section; compression 10,000 lb. per sq. in. on gross section; shop rivets 10,000 lb. per sq. in. for shear, and 20,000 lb. per sq. in. for bearing; field rivets 80 per cent of shop rivets. The bell shaped flare was 50 ft. high. The stack was lined with a 4 in. brick lining supported by horizontal angles riveted to the inside of the shell and spaced 15 ft. apart.

The Babcock and Wilcox Co. uses allowable stresses as given in Table VI.

The Chicago Bridge and Iron Works designs for compression on the leeward side, with 12,000 lb. per sq. in. compression on net section, and 14,000–125 d/t lb. per sq. in. compression on gross section with a maximum of 10,000 lb. per sq. in.

Sargent and Lundy, consulting engineers, specify a wind pressure of 25 lb. per sq. ft., and a tensile stress of 12,000 lb. on net section.

On account of corrosion the working stresses used in designing the plates and joints should be kept low. The author would recommend that the same allowable stresses for designing steel stacks be used as are used in designing steel water tanks in Chapter XI. With these conservative stresses it would not appear to be necessary to consider the dead load stresses due to the weight of the steel stack and the stack lining unless these dead load stresses exceed the unit stresses due to wind pressure by more than 10 per cent. This makes it unnecessary to consider the weight of the steel in an unlined stack under 200 ft. in height. The allowable unit stresses recommended by the author are given in § 4, of the specifications in the latter part of this chapter.

The pressures of bearing plates on masonry should not exceed the following in lb. per sq. in.

Brick masonry laid in cement mortar	200
Portland cement concrete, 1–2–4	400
First-class sandstone	300
First-class limestone	400
First-class granite	600

Compressive Stresses.—There will be tensile stresses in the plates on the windward side, and compressive stresses in the plates on the leeward side of the stack. The efficiency of the joint on the compressive side will depend upon the strength of the rivet in shear and in bearing of the rivet on the plate, and not on the strength of the plate in tension. The load on the leeward side will be increased by the weight of the lining. The maximum stresses on the rivets will come on the leeward side and the plates should be designed for these stresses.

To prevent flattening on the windward side or buckling on the leeward side it is common to also design the plates for a compressive stress on the gross section of the plates using a lower stress on gross area than on net area of the plate. The Chicago Bridge & Iron Co. requires that the stress on the gross area be not greater than $f_c = 14{,}000 - 125d/t$. For $\frac{1}{4}$ in. plates and a diameter of 8 ft., this gives $f_c = 10{,}000$ lb. per sq. in.; and for a 16 ft. diameter stack $f_c = 6{,}000$ lb. per sq. in. The Babcock and Wilcox Co. specifies that the thickness of the plate in any course shall not be less than the diameter divided by 500.

Horizontal Seams.—Horizontal seams for plates $\frac{3}{8}$ in. and less in thickness are commonly made with lap joints. Single riveted lap joints may be used for the upper section, where it is not necessary to develop the strength of the joints. From Table IV it will be seen that lap joints with two lines of rivets should be used with plates $\frac{5}{16}$ to $\frac{7}{16}$ in. thick, and three lines of rivets with plates $\frac{1}{2}$ to $\frac{5}{8}$ in. thick. Butt joints should preferably be used for plates thicker than $\frac{5}{8}$ in. All joints should be caulked on the inside. The rivet spacing along the caulked edge of a plate should not be greater than 10 times the thickness of the plate. The rivet spacing should not be less than $2\frac{1}{2}$ times the thickness of the plate.

Vertical Seams.—The rivets in the vertical seams, must be spaced sufficiently close to prevent buckling and permit caulking. The shearing on the rivets in the vertical joint is small and may

be neglected. The vertical joints are usually made single lap with the same spacing as the horizontal joints. The rivet spacing for caulked joints is given in Table 113, Part II. Table 113 is calculated for water tanks. For stacks this spacing may be slightly exceeded. For efficiency of riveted lap joints see Table IV.

Efficiency of Riveted Joints.—Formulas for calculating the efficiency of riveted joints are given in Chapter XVII. The properties of lap joints are given in Table IV. The properties of lap joints and butt joints are given in Table IIa, Chapter XI. From the tables of properties of joints it will be seen that the efficiency of single riveted lap joints varies from 40 to 50 per cent, while for lap joints with two rows of rivets it is possible to obtain an efficiency of 70 per cent.

COST OF STEEL STACKS.—The cost of the steel plates and structural material may be obtained from the ENGINEERING-NEWS-RECORD or the IRON AGE. The approximate weight of the steel in steel stacks may be taken from Table V. The shop cost of steel stacks will be practically the same as for flat bottom steel tanks, for the shop cost (1913) of which see Table V, Chapter XIII. In 1916 the actual cost of detailing and erecting several steel stacks in the middle west was as follows. A steel stack 18 ft. diameter and 200 ft. high, weighing 117 tons, cost $22.00 per ton to erect. A steel stack 10 ft. diameter and 200 ft. high, weighing 61 tons, cost $32.00 per ton to erect. A steel stack 12 ft. diameter and 150 ft. high, weighing 48 tons, cost $1.00 per ton to detail, and $20.00 per ton to erect. A steel stack 10 ft. diameter and 188 ft. high, weighing 67 tons, cost 75 cents per ton to detail, and $28.00 per ton to erect.

TABLE IV.

PROPERTIES OF RIVETED LAP JOINTS.

Tensile stress in plates 12,000 lb. Shear in rivets 9,000 lb. Bearing on plates 18,000 lb.

Thickness of Plate, in.	Number of Rows of Rivets	½ in. Rivet.			⅝ in. Rivet.			¾ in. Rivet.			⅞ in. Rivet.		
		Efficiency of Joint per cent	Pitch of Rivets, in.	Effective Section of Plate, in.	Efficiency of Joint per cent	Pitch of Rivets, in.	Effective Section of Plate, in.	Efficiency of Joint per cent	Pitch of Rivets, in.	Effective Section of Plate, in.	Efficiency of Joint per cent	Pitch of Rivets, in.	Effective Section of Plate, in.
¼	1	39.3	1.50	.098	48.7	1.88	.121						
	2	65.4	1.80	.163	70.7	2.50	.177						
5/16	1	31.4	1.50	.098	39.5	1.88	.124	47.1	2.25	.147			
	2	60.0	1.57	.187	65.4	2.50	.205	70.5	3.00	.220			
⅜	1				32.7	1.88	.123	39.3	2.25	.147	45.8	2.63	.172
	2				61.5	2.00	.231	66.6	2.63	.250	70.4	3.38	.264
7/16	1							33.7	2.25	.147	39.3	2.63	.172
	2							63.5	2.38	.279	67.1	3.00	.294
½	2							58.9	2.25	.295	64.4	2.80	.322
	3							69.5	2.88	.347	72.9	3.71	.364
9/16	2							52.4	2.25	.295	61.0	2.63	.344
	3							66.9	2.63	.376	70.5	3.38	.397
⅝	2							47.1	2.25	.294	55.0	2.63	.344
	3							64.5	2.47	.403	68.5	3.16	.428

Note. Distance between rivets at caulked edges shall not exceed 10 times thickness of plate. Distance from centers of rivets to edge of plate shall be 1 in. for ½ in. rivets, 1⅛ in. for ⅝ in. rivets, 1¼ in. for ¾ in. rivets, 1½ in. for ⅞ in. rivets. Distance between rows of rivets shall be 1½ in. for ½ in. rivets, 1¾ in. for ⅝ in. rivets, 2 in. for ¾ in. rivets, 2¼ in. for ⅞ in. rivets.

EXAMPLES OF STEEL STACKS.—The following examples of steel stacks will show many important details and standard practice in the design of self-supporting steel stacks.

Steel Stack for University of Colorado.—The self-supporting steel stack 150 ft. by 7 ft., shown in Fig. 2, was built at the University of Colorado in 1910. The diameter of the base of the flare is 11 ft. $8\frac{3}{4}$ in. The stack is anchored to the masonry foundation by 10 anchor bolts $2\frac{1}{4}$ in. in diameter by 33 ft. long. The anchor bolts are attached to the inside of the shell by means of a bracket made of two angles 5 in. by $3\frac{1}{2}$ in. by $\frac{1}{2}$ in. by 2 ft. 9 in. long. The stack is lined with fire brick, to a height of 28 ft. 6 in. For the lower 18 ft. the brick is 8 in. thick, and for the remaining 10 ft. 6 in. the lining is 4 in. thick. The upper 102 ft. 6 in. of the stack is lined with vitrobestos, 2 in. thick. The vitrobestos lining is supported on 4 in. by 4 in. by $\frac{5}{16}$ in. angles spaced 3 ft. vertically and riveted to the stack plates.

Steel Stack for Smokeless Powder Plant.—The details of the 250 ft. by 16 ft. self-supporting steel stack built at the U. S. Government Explosives Plant at Nitro, West Virginia, are shown in Fig. 3. The stack was designed for a wind load of 25 lb. per sq. ft. on the projected area. The stack has a straight flare 30 ft. high, the diameter of the base being 20 ft. The stack was anchored to the concrete foundation by 28 anchor bolts $2\frac{1}{2}$ in. in diameter and 28 ft. $2\frac{1}{2}$ in. long. The diameter of the anchor bolt circle was 20 ft. $7\frac{1}{2}$ in. The details of the riveted joints are given in Fig. 3. Five-eighths in. rivets were used with $\frac{1}{4}$ in. plates, $\frac{3}{4}$ in. rivets with $\frac{5}{16}$ in. and $\frac{3}{8}$ in. plates, and $\frac{7}{8}$ in. rivets with $\frac{7}{16}$ in., $\frac{1}{2}$ in. and $\frac{9}{16}$ in. plates. The plates were medium open hearth steel with an ultimate strength of 55,000 lb. per sq. in. The stack plates were designed for an allowable tensile stress of 11,000 lb. per sq. in., and a compressive stress of 16,000 lb. per sq. in. Allowable shear in rivets 9,000 lb. per sq. in., allowable bearing of rivets on plates 18,000 lb. per sq. in. The stack was unlined. The stack was carried on a reinforced concrete foundation which contained the flue.

Steel Stack for Minneapolis General Electric Company.—The details for the 201 ft. by 16 ft. 5 in. self-supporting steel stack designed by the H. M. Byllesby Company for the Riverside Station of the Minneapolis General Electric Company, Minneapolis, Minnesota, are given in Fig. 4 and Fig. 5.

This stack was designed for a wind load of 30 lb. per sq. in. on the projected area of the stack. The lower 49 ft. of the stack is lined with No. 2 radial fire brick, $4\frac{1}{2}$ in. thick with 1 in. of cement grout, while the upper 152 ft. of the stack is lined with a concrete lining 4 in. thick. The concrete is reinforced with No. 26 triangular mesh, and is supported at each horizontal joint by 3 in. by 3 in. by $\frac{3}{8}$ in. angles. The concrete was a 1–2–4 Portland cement concrete. The aggregate was stone crushed to pass a $\frac{3}{4}$ in. screen. The fire brick were laid up in the best quality fire clay, mixed with water to form a thin paste. The brick were dipped in this paste and were well rubbed into place to make a perfectly tight joint. Common brick laid in cement mortar were used for the outside lining in the lower part of the bell.

The steel stack is supported on four braced columns, connected at the top by four main girders and six auxiliary girders.

The base of the stack is anchored to the supporting girders by means of 16 anchor bolts 3 in. in diameter. The anchor bolts are placed in pairs, one inside and the other outside the steel shell.

The steel ladder has main vertical wrought iron bars $2\frac{1}{2}$ in. by $\frac{1}{2}$ in., extending from the bottom to the top of the stack, with $\frac{3}{4}$ in. round rungs spaced 12 in. apart. The ladder is fastened to the stack by means of ladder lugs spaced 10 ft. apart. These lugs are made of $2\frac{1}{2}$ in. by $\frac{1}{2}$ in. bars and are braced on the outside by means of two $2\frac{1}{2}$ in. by $\frac{3}{8}$ in. bars extending from the bottom to the top of the stack. The ladder lugs give a clear space of 21 in. by 24 in. and serve as a guard for a man climbing the ladder.

The details of the riveting are shown in Fig. 4. The horizontal joints are double riveted for the lower 191 ft. of the stack and single riveted for the upper 90 ft. of the stack. The vertical joints are double riveted for the lower 191 ft. of the stack and single riveted for the upper 90 ft. of the stack.

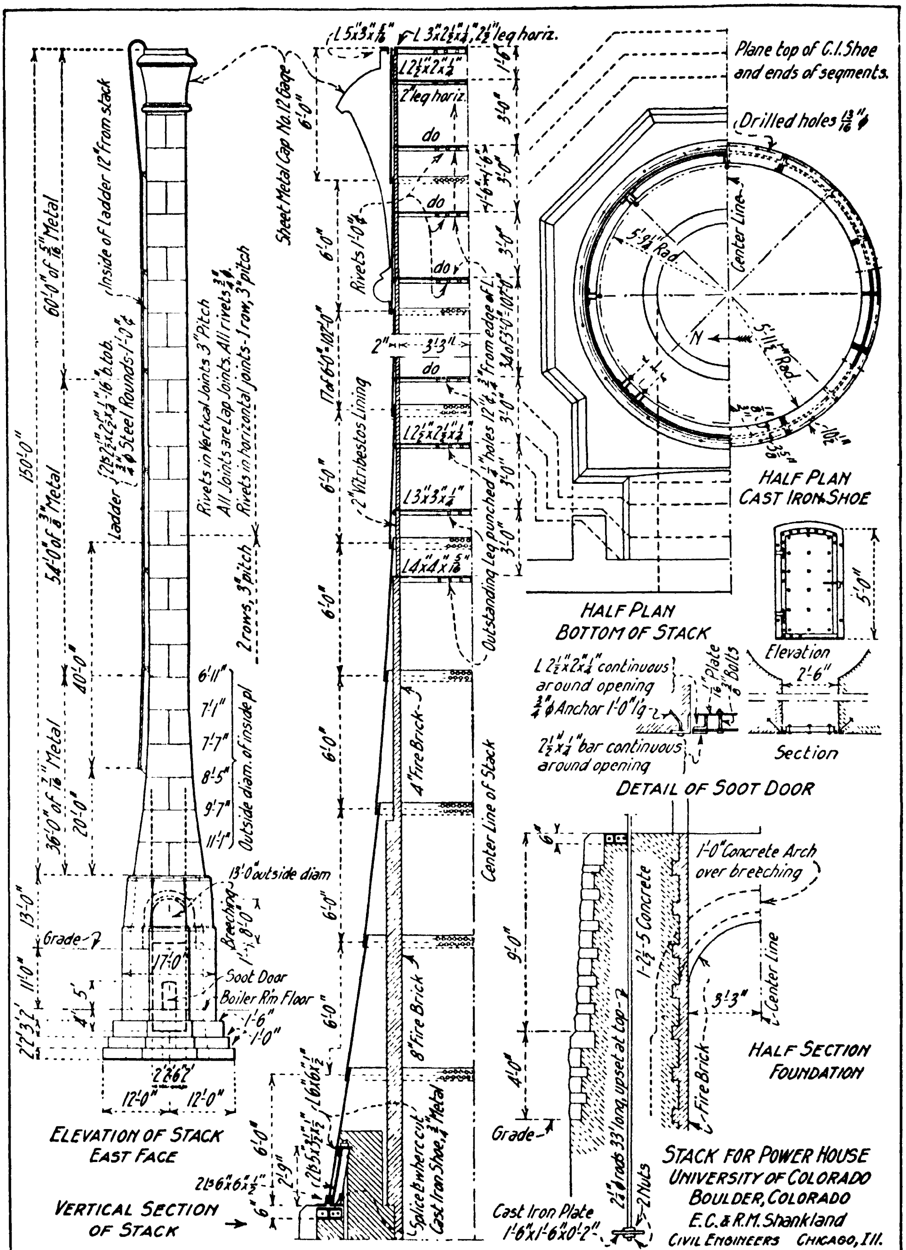


FIG. 2. STEEL STACK FOR UNIVERSITY OF COLORADO.

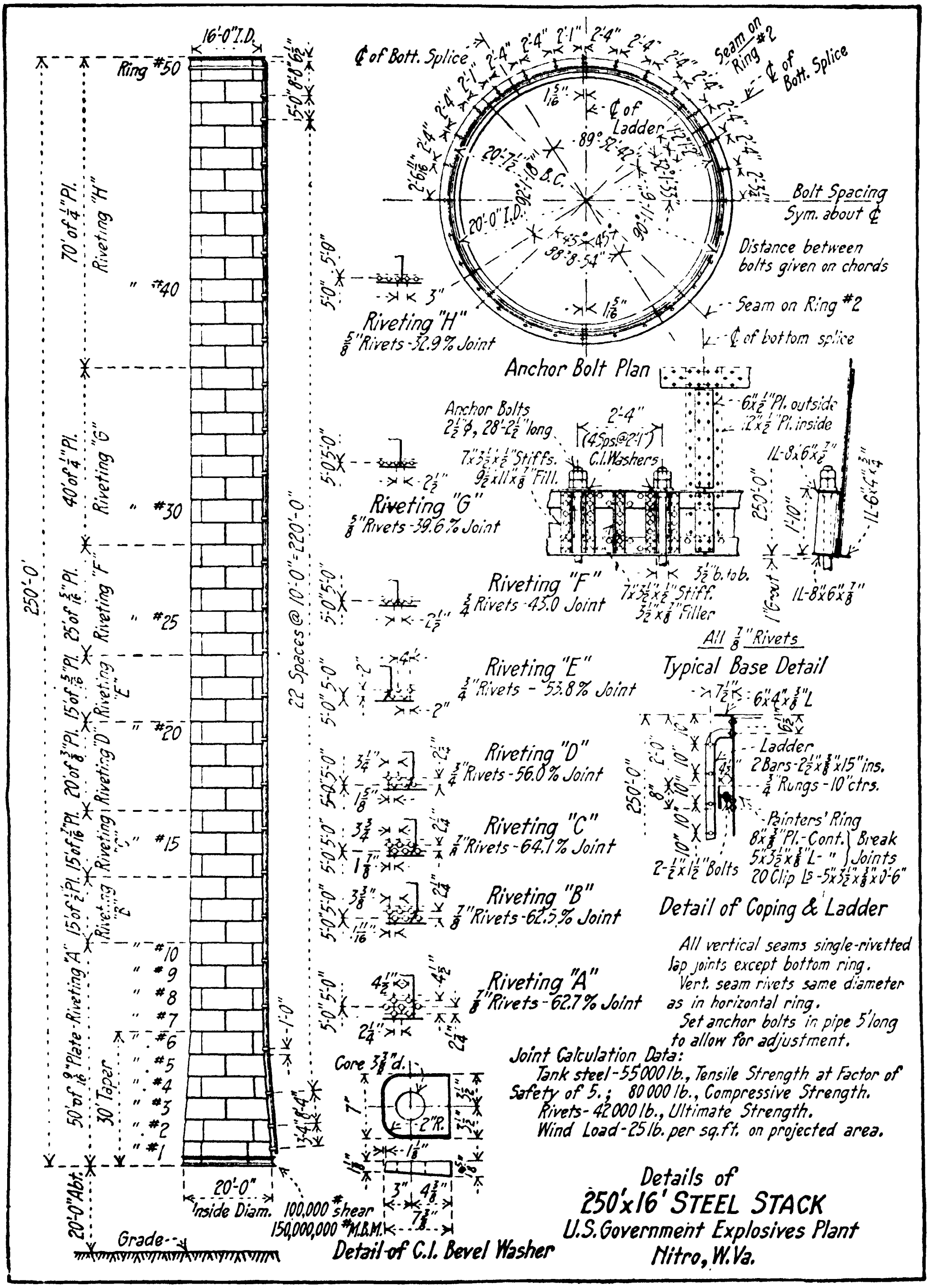


FIG. 3. STEEL STACK FOR SMOKELESS POWDER PLANT.

The details of the painter's trolley are shown in Fig. 4 and the cast iron cap is shown in Fig. 5. The steel plates and other details were painted in the shop with one coat of red lead paint, and were given two coats of paint after erection.

The Babcock and Wilcox Self-supporting Steel Stacks.—The self-supporting steel stacks, the data for which are given in Table V, were designed for a wind load of 25 lb. per sq. ft. on the projected area of the stack. The stacks have a straight conical flare at the base, the apex

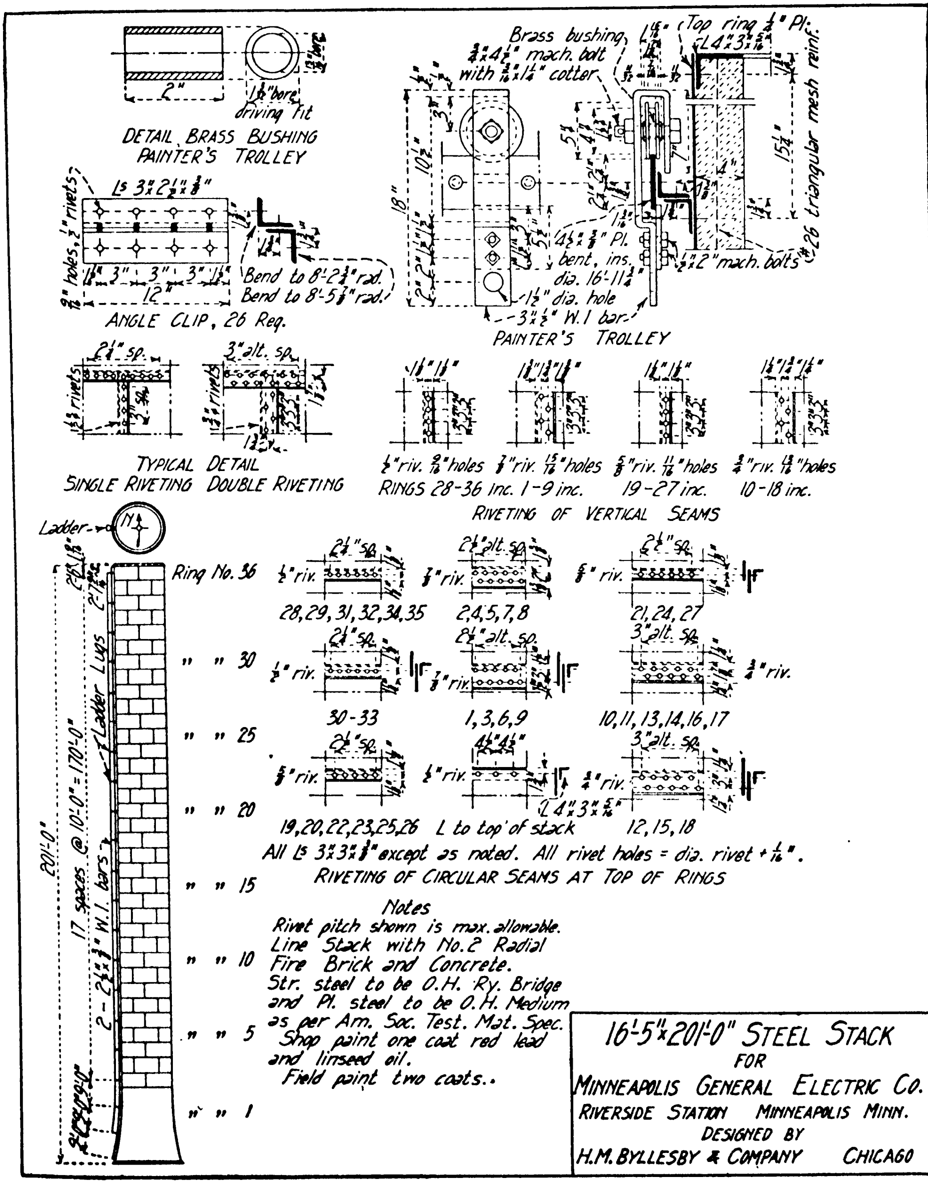


FIG. 4. STEEL STACK FOR MINNEAPOLIS GENERAL ELECTRIC CO.

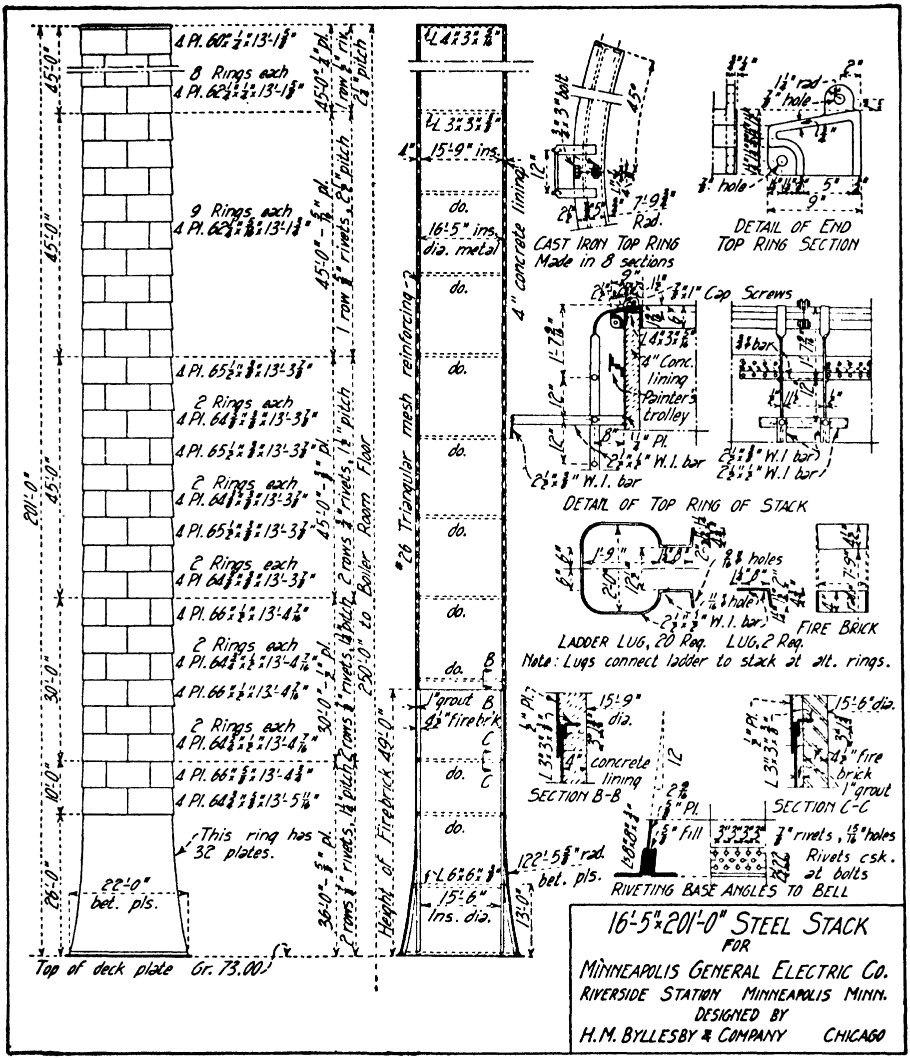


FIG. 5. STEEL STACK FOR MINNEAPOLIS GENERAL ELECTRIC CO.

of the cone being at the top of the stack, and the flared part being approximately one-fourth the height of the stack. This makes the diameter at the base approximately one-third larger than the diameter of the straight part of the stack. The flared part of the stack is built of plates of a uniform thickness. The flare also gives a larger diameter for the flue openings and makes a more easy flow of gases. The flared stack saves from 10 to 15 per cent over straight stacks of equal height.

The stacks in Table V were assumed as unlined and the weight of the steel was neglected in making the calculations. The allowable stresses, and rivet spacing used in the design are given in Table VI.

TABLE V.

DATA FOR SELF-SUPPORTING STEEL STACKS.

(The Babcock and Wilcox Company.)

Diam.	Height	Horse Power	Concrete Foundation (circular)			Anchor Bolts		Bottom Section Including Flare		2nd Section		3rd Section		4th Section		5th Section		6th Section		7th Section		Flare Straight Conical		Total Weight
			Diam.	Height	Earth Pressure	Number	Diam.	Height	Plates	Height	Plates	Height	Plates	Height	Plates	Height	Plates	Height	Plates	Height	Plates	Diam. Base	Height	
ft.	ft.		ft.	ft.	lb.		in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft., in.	ft.	lb.
4	100	348	17.6	5.8	1,875	16	1 3/8	45	5/16	15	1/4	40	3/16									5–9	30	20,000
5	125	632	20.8	6.9	2,210	14	1 7/8	45	3/8	15	5/16	20	1/4	45	3/16							6–7	30	32,800
6	125	934	21.8	7.2	2,300	24	1 3/8	40	5/16	15	5/16	20	1/4	50	3/16							8–10	40	35,690
7	150	1,418	24.8	8.2	2,620	18	1 7/8	55	3/8	20	5/16	20	1/4	55	3/16							9–7	40	55,740
8	175	2,027	27.7	9.2	2,940	20	2 1/8	55	7/16	20	3/8	20	5/16	20	1/4	60	3/16					10–4	40	80,950
9	200	2,771	30.5	10.2	3,260	24	2 1/8	75	7/16	20	3/8	20	5/16	25	1/4	60	3/16					12–5	55	108,900
10	200	3,448	31.3	10.4	3,330	26	2 1/8	70	7/16	20	3/8	20	5/16	25	1/4	65	3/16					13–3	50	119,000
10	250	3,855	33.3	11.1	3,530	26	2 3/4	80	9/16	20	1/2	20	7/16	20	3/8	20	5/16	25	1/4	65	3/16	13–4	55	187,400
12	225	5,330	34.7	11.6	3,720	32	2 1/8	80	7/16	20	3/8	25	5/16	100	1/4							16–4	60	165,000
12	250	5,618	36.6	12.2	3,900	30	2 1/2	85	1/2	20	7/16	20	3/8	25	5/16	100	1/4					16–3	65	206,800
14	225	7,310	36.1	12.0	3,840	40	1 7/8	90	3/8	25	5/16	110	1/4									19–8	65	173,400
14	275	8,110	39.9	13.3	4,260	36	2 1/2	75	1/2	20	1/2	20	7/16	25	3/8	25	5/16	110	1/4			19–3	75	265,600
15	225	8,440	36.8	12.3	3,940	42	1 7/8	85	3/8	140	5/16											20–6	60	197,300
15	275	9,340	40.6	13.5	4,320	36	2 1/4	85	1/2	25	7/16	25	3/8	140	5/16							19–8	65	288,500
16	250	10,138	39.3	13.1	4,130	42	2 1/8	55	7/16	30	7/16	25	3/8	145	5/16							20–6	55	250,300
16	300	11,105	43.1	14.4	4,610	38	2 3/4	60	9/16	20	9/16	25	1/2	25	7/16	25	3/8	145	5/16			20–0	60	357,500
18	250	12,894	40.5	13.5	4,320	50	1 7/8	70	3/8	30	3/8	150	5/16									25–0	70	262,800
18	300	14,123	44.4	14.8	4,740	44	2 1/2	70	1/2	25	1/2	25	7/16	30	3/8	150	5/16					23–6	70	375,300
20	250	15,980	41.6	13.3	4,450	54	1 7/8	60	3/8	190	3/8											26–4	60	323,700
20	300	17,505	45.5	15.2	4,860	48	2 1/2	60	1/2	25	1/2	25	7/16	190	3/8							25–0	60	442,600

The thickness of the plate in any course is kept not less than 1/500 the diameter of the stack, so that the tendency to flatten the stack will not produce a stress greater than 20,000 lb. per sq. in.

The base plates are wrought steel plates 12 in. x $\frac{5}{8}$ in. to 24 in. x $1\frac{1}{4}$ in. and are provided under the edge of the shell and the anchor bolt angles. The base plates are designed for a bending stress of 20,000 lb. per sq. in. in the plates, and a bearing of 220 lb. per sq. in. on the concrete foundation.

One diameter of anchor bolt is used with each thickness of plate. The anchor bolts are upset, and are designed for a stress of 16,000 lb. per sq. in. Anchor bolts are arranged in pairs, one inside and one outside of the stack and bear on the ends of pairs of vertical angles, fastened to the shell by ivets in double shear. Each pair of anchor bolts is anchored to a pair of channels approximately one foot above the bottom of the concrete foundation.

The top ring is made of 3 in., 4 in. or 6 in. Z-bars, which are used for a painter's trolley.

The standard ladder has $\frac{3}{4}$ in. rounds and $2\frac{1}{2}$ in. by $\frac{1}{2}$ in. side bars, and extends to the top of the stack.

A cleaning door 18 in. by 24 in. is used on all stacks.

TABLE VI.

DATA FOR CALCULATION OF SELF-SUPPORTING STEEL STACKS.

(The Babcock and Wilcox Company.)

Stress on horizontal section, lb.	Plate Thickness, In.	Diameter Rivets, In.	Pitch Rivets, In.	Stress at Rivets, lb. sq. in.			
				Bearing on Rivet	Shear on Rivet	Tension in Plate	Tension in Gross Area Plate
600	$\frac{3}{16}$	$\frac{3}{8}$	2	15,800	9,270	4,020	3,200
1,100	$\frac{1}{4}$	$\frac{1}{2}$	2	16,600	9,950	5,980	4,400
1,700	$\frac{5}{16}$	$\frac{3}{8}$	2	16,600	10,000	8,080	5,440
2,360	$\frac{3}{8}$	$\frac{3}{4}$	(2 Rows) 4	16,100	9,860	7,830	6,295
3,100	$\frac{7}{16}$	$\frac{3}{4}$	(2 Rows) 3	13,600	9,750	9,600	7,090
3,930	$\frac{1}{2}$	$\frac{7}{8}$	(2 Rows) 3	13,000	9,150	11,400	7,690
4,800	$\frac{9}{16}$	1	(2 Rows) $3\frac{1}{2}$	14,480	10,050	12,100	8,545

Steel Stack for Gary Tube Mills.—The self-supporting steel stack, 6 ft. 2 in. diameter and 125 ft. high shown in Fig. 6, was built by the American Bridge Company for the Gary Tube Mills, Gary, Indiana. The plates were made of copper bearing steel containing from 0.25 to 0.30 per cent copper. The plates are made thicker than required by the calculations on account of danger of corrosion. The stack is lined with a 6 in. brick lining with a 1 in. air space between the brick and the steel shell.

The stack has a straight conical flare at the bottom 12 ft. 6 in. high. The diameter of the base of the flare is 10 ft. 4 in., the ratio of the diameter of the base to the diameter of the straight stack being as 5 to 3.

The stack is anchored to the foundation by means of 8 anchor bolts $2\frac{1}{4}$ in. diameter and 18 ft. long. The anchor bolts are placed on the outside of the shell in pairs spaced 18 in. apart, the diameter of the anchor bolt ring being 11 ft. 2 in.

The steel ladder is provided with safety rungs spaced 2 ft. apart. The main ladder supports are angles $2\frac{1}{2}$ in x $2\frac{1}{2}$ in. x $\frac{5}{16}$ in., and the rungs are $2\frac{1}{2}$ in. by $\frac{5}{16}$ in. bars spaced 12 in. apart. The vertical angles are fastened to the steel shell by means of angle lugs spaced about 12 ft. apart.

All joints except the base have two rows of rivets with a staggered pitch of not less than 3 in.

The steel plates were given a shop coat of iron oxide paint, and a field coat of black graphite paint.

The base plate is made of cast iron, cast in two sections. The cast iron cap was cast in one section.

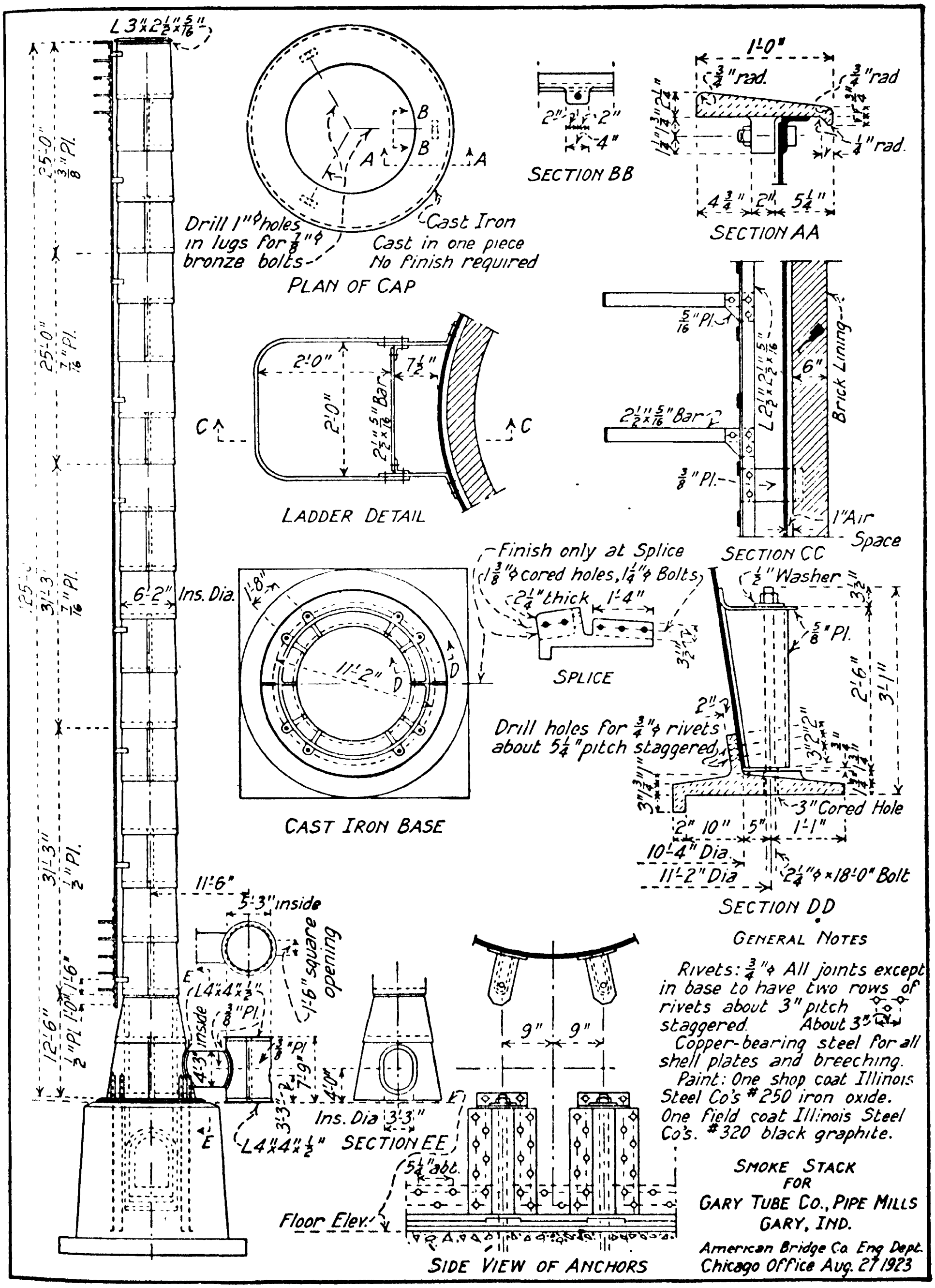


FIG. 6. STEEL STACK FOR GARY TUBE MILLS.

GENERAL SPECIFICATIONS FOR SELF-SUPPORTING STEEL STACKS.

BY

MILO S. KETCHUM,

M. Am. Soc. C. E.

1924.

1. **Definition.**—A self-supporting steel stack is a chimney made of steel plates that is supported on a foundation, the wind loads being transmitted directly to the foundation by cantilever action. Self-supporting steel stacks are commonly cylindrical. To give a larger base and to permit better entrance of flue gases the lower part of a steel stack is commonly given a conical flare. The economical ratio of the diameter of the cylindrical stack to the diameter of the base of the flare will ordinarily vary from $\frac{1}{3}$ to $\frac{5}{4}$. The plates in the conical or bell mouth flare shall not be thinner than the thickness of the lower course in the cylindrical stack. Self-supporting steel stacks may be lined or unlined.

2. **Loads.**—The dead load shall consist of the weight of the plates and structural steel, the top ring and ladder, the angles to support the lining, and the weight of the lining with its reinforcement.

3. The wind pressure shall be assumed as 25 lb. per sq. ft. acting horizontally in any direction on the projection of the stack.

4. **Unit Stresses.**—All parts of the structure shall be proportioned so that the maximum stresses, except as specified in § 6 shall not exceed the following in lb. per sq. in.

Tension on stack plates, net section	12,000
Tension in anchor bolts and other structural parts, net section	16,000
Compression on stack plates, gross section	10,000
Shear on shop rivets	12,000
Shear on field rivets	9,000
Shear in plates	10,000
Bearing pressure on shop rivets and plates	24,000
Bearing pressure on field rivets	18,000
Bending stresses on shapes and built girders	16,000
Bending stresses in pins	24,000

5. For compression members the allowable stress shall be given by the formula

$$p = 16{,}000 - 70l/r \qquad (1)$$

where p = allowable stress in lb. per sq. in., l = length of the member and r = least radius of gyration of the section, both in inches. For compression members the ratio l/r shall not exceed 125 for main members and 200 for secondary members.

6. In combining the wind load stresses and the dead load stresses due to the weight of the stack and the lining, the dead load stresses may be neglected in designing the stack plates and joints if the dead load stresses are not more than 10 per cent of the stresses due to wind load. If the dead load stresses in the plates and joints exceed 10 per cent of the wind stresses, the sum of the stresses due to dead and wind loads shall be used, but with allowable tresses 10 per cent in excess of those permitted when dead load stresses are not considered, but the sections shall not be less than when dead load is not considered.

7. The compressive stresses in plates due to wind pressure and dead loads shall not exceed 10,000 lb. per sq. in. on the gross section of the plate, see § 6.

8. **Pressures on Masonry.**—The pressures of bearing plates on masonry shall not exceed the following in lb. per sq. in.

Brick masonry laid in cement mortar	200
Portland cement concrete, 1–2–4	400
First-class sandstone	300
First-class limestone	400
First-class granite	600

9. **Details of Construction.**—The plates forming the sides of the stack shall have the upper diameter less than the lower diameter so that each course shall telescope over the course below it.

10. **Vertical Joints.**—The vertical joints in the cylindrical stack and conical flare shall pre-

ferably be lap joints. The vertical joints in the bell mouth flare of large stacks shall be butt joints. Vertical butt joints shall be used on plates having horizontal butt joints.

11. **Horizontal Joints.**—For horizontal joints single-riveted lap joints shall preferably be used for $\frac{1}{4}$ in. plates, double-riveted lap joints for $\frac{5}{16}$ in. $\frac{3}{8}$ in., and $\frac{7}{16}$ in. plates, and triple-riveted lap joints for $\frac{1}{2}$ in., $\frac{9}{16}$ in. and $\frac{5}{8}$ in. plates. Butt joints should be used for plates thicker than $\frac{5}{8}$ in.

12. **Rivets.**—Rivets $\frac{5}{8}$ in. diameter shall preferably be used for $\frac{1}{4}$ in. plates, rivets $\frac{3}{4}$ in. diameter shall be used for $\frac{5}{16}$ in. and $\frac{3}{8}$ in. plates, $\frac{7}{8}$ in. rivets shall be used for $\frac{1}{2}$ in. $\frac{9}{16}$ and $\frac{5}{8}$ in. plates. Rivets 1 in. in diameter shall be used for plates thicker than $\frac{5}{8}$ in.

Rivets shall be spaced so as to make the most economical seams. A table of riveted joints is given in Table IV.

13. In no case shall the spacing of the rivets along the caulked edges of plates be more than 10 times the thickness of the plates. The rivet spacing shall never be less than $2\frac{1}{2}$ times the diameter of the rivet.

14. Plates more than $\frac{5}{8}$ in. thick and not more than $\frac{7}{8}$ in. thick shall be sub-punched with a punch $\frac{3}{16}$ in. smaller than the nominal size of rivet, and shall be reamed to a diameter $\frac{1}{16}$ in. larger than the rivet. Plates thicker than $\frac{7}{8}$ in. thick shall be drilled.

15. **Minimum Sections.**—The minimum thickness of plates in the stack shall be $\frac{1}{4}$ in. and preferably stack plates shall not be less than $\frac{5}{16}$ in. thick.

16. **Caulking.**—All stack plates shall be sheared or planed to a proper bevel for caulking.

17. All stack plates shall be caulked from the inside of the stack, and with a round-nosed caulking tool. The use of foreign material for caulking will not be permitted.

18. **Painter's Trolley Track.**—Near the top of the stack there shall be provided a Z-bar to act as a track for the painter's trolley, and to stiffen the top of the stack. The section modulus of this stiffening ring shall not be less than $d^2/250$, where d = diameter of stack in feet.

19. **Stiffening Angles.**—On large stacks or where n is less than 25 as calculated by formula 2, circular stiffening angles shall be provided in order to prevent buckling due to wind pressure. The distance between angles shall not be less in feet than

$$n = 900t^{1/2}/d \tag{2}$$

where t = thickness of plate in inches and d = diameter of stack in feet.

20. **Cap Ring.**—The top of the stack shall generally have a cast iron cap ring. The ring shall be bolted to the top of the stack with non-corrosive bolts.

21. **Ladder.**—There shall be a ladder 1 ft. 3 in. wide extending from a point 8 ft. above the foundations to the top of the stack. Each ladder shall be made of $2\frac{1}{2}$ in. x $\frac{3}{8}$ in. bars with $\frac{3}{4}$ in. round rungs 1 ft. apart. Ladders on stacks more than 100 ft. in height shall be provided with safety rings made of 2 in. x $\frac{3}{8}$ in. bars, braced on the outside with one 2 in. x $\frac{3}{8}$ in. vertical bar. These rings shall have an inside clearance of not less than 24 in. in width and depth, and shall be spaced not more than 10 ft. apart vertically.

22. **Anchor Bolts.**—The size and number of anchor bolts shall be determined by the maximum uplift due to wind pressure. The anchor bolts shall be not less than $1\frac{1}{2}$ in. in diameter and shall be set deep enough to take the necessary uplift. Anchor bolts shall preferably be in pairs, one on the inside and the other on the outside of the steel shell, both fastened to the stack plates with plate and angle connections. The lower ends of anchor bolts shall be provided with an anchor plate not less than $\frac{1}{2}$ in. thick. Double anchor bolts shall have channel anchors.

23. **Flue Connections.**—The connections for flues and breeching shall be reinforced with plates and angles so that the strength of the stack section shall not be reduced.

24. **Cleanout.**—Where a cleanout is not provided in the foundation a cleanout door 18 in. by 24 in. shall be provided near the bottom of the stack. The opening for the cleanout door shall be properly reinforced.

25. **Base Plates.**—The lower stack plates shall be connected to a bottom flange made of built plates and angles, or of cast steel. The flanges shall be riveted to the vertical plates and shall be fully bedded on the masonry.

26. **Lining.**—The stack shall be lined or unlined as specified on the plans. If lined the lining shall be supported on curved angles riveted at the horizontal joints. The horizontal angles shall be spaced vertically at distances not greater than one-half the diameter of the stack. Brick lining shall ordinarily be 4 in. thick. Fire brick shall be laid in fire clay while ordinary brick shall be laid in cement mortar. The space between the brick work and the steel shell shall be grouted with 1 to 2 Portland cement mortar.

27. **Materials.**—The steel for plates and shapes and bars shall be made by the open-hearth process and shall comply with the Specifications for Structural Steel for Buildings of the American Society for Testing Materials. If specified the steel for plates and shapes shall contain 0.30 per cent copper.

28. **Workmanship.**—The workmanship and finish shall be equal to the best in modern shop practice.

29. All material shall be thoroughly straightened in the shop by methods that will not injure it, before being laid off or worked in any way.

30. The shearing shall be neatly done and all portions exposed to view shall have a neat and workmanlike finish.

31. The size of each rivet shall be understood to be the actual size of the cold rivet before it is heated.

32. All plates and shapes shall be shaped to the proper curves by cold rolling; heating or hammering for straightening or curving will not be allowed.

33. Plates to be scarfed may be heated to a cherry red color, but not hot enough to ignite a piece of dry wood when applied to it.

34. All plates and shapes shall be punched before being bevel-sheared for caulking.

35. The diameter of the die used in punching rivet holes shall not exceed that of the punch by more than $\frac{1}{16}$ in.

36. All punched and reamed holes shall be clean cut without torn or ragged edges.

37. Rivet holes shall be accurately spaced; poorly matched holes if not sufficient for rejection shall be reamed and a larger rivet used in the hole thus reamed.

38. The use of drift pins will be allowed only for bringing the parts of the structure together. Sufficient force shall not be used to enlarge rivet holes by drifting.

39. Plates and other parts to be riveted shall be closely drawn together before driving the rivets. In stack plates not less than one-third of the holes shall be filled with erection bolts well drawn up before driving the rivets.

40. Rivets shall be driven by power tools wherever possible. Pneumatic hammers shall be used in preference to hand driving. All rivet heads shall be concentric with the holes.

41. All caulking shall be done with a round nosed tool, and only by experienced and skilled men. Caulking around rivet holes will not be allowed All loose rivets shall be cut out and be redriven. All fractured material shall be replaced free of cost to the purchaser.

42. The inspector shall have free access at all times to all parts of the structure where the material is being fabricated. If the inspector through oversight or otherwise has accepted material or work which is defective or contrary to these specifications, this material no matter in what stage of completion may be rejected by the purchaser.

43. **Painting.**—Before leaving the shop all steel work shall receive one coat of approved paint or boiled linseed oil mixed with one ounce of lampblack to each gallon of oil.

44. After the structure is erected and all seams are caulked, the steel work shall be painted both inside and outside with two coats of approved paint. Painting done in the open air shall never be done in wet or freezing weather.

45. **Masonry Foundations.**—The allowable pressure on firm clay or gravel should not ordinarily exceed 3,000 lb. per sq. ft. In all cases a thorough examination should be made of the ground and site before designing or constructing the foundations. For high self-supporting steel stacks if there is any question about the bearing power of the soil, the masonry should be carried on piles.

All foundations shall be carried well below frost line, and the anchor bolts shall be placed deep enough to develop their full strength.

46. Foundations shall be made of 1 part Portland cement, 2 parts sand and 4 parts gravel or broken stone. The concrete shall be mixed and placed in accordance with the most approved practice.

47. If the self-supporting steel stack is supported on a steel substructure, the latter shall conform to Ketchum's "Specifications for Steel Frame Buildings," Chapter I.

CHAPTER XII.

STRUCTURAL DRAFTING.

PLANS FOR STRUCTURES.

Introduction.—The plans for a structure must contain all the information necessary for the design of the structure, for ordering the material, for fabricating the structure in the shop, for erecting the structure, and for making a complete estimate of the material used in the structure. Every complete set of plans for a structure must contain the following information, in so far as the different items apply to the particular structure.

In writing this chapter the instructions of many bridge companies have been consulted; special credit being due the instructions prepared by the American Bridge Company, the Pennsylvania Steel Company, and the McClintic-Marshall Construction Company.

1. **General Plan.**—This will include a profile of the ground; location of the structure; elevations of ruling points in the structure; clearances; grades; (for a bridge) direction of flow, high water, and low water; and all other data necessary for designing the substructure and superstructure.

2. **Stress Diagram.**—This will give the main dimensions of the structure, the loading, stresses in all members for the dead loads, live loads, wind loads, etc., itemized separately; the total maximum stresses and minimum stresses; sizes of members; typical sections of all built members showing arrangement of material, and all information necessary for the detailing of the various parts of the structure.

3. **Shop Drawings.**—Shop detail drawings should be made for all steel and iron work and detail drawings of all timber, masonry and concrete work.

4. **Foundation or Masonry Plan.**—The foundation or masonry plan should contain detail drawings of all foundations, walls, piers, etc., that support the structure. The plans should show the loads on the foundations; the depths of footings; the spacing of piles where used; the proportions for the concrete; the quality of masonry and mortar; the allowable bearing on the soil; and all data necessary for accurately locating and constructing the foundations.

5. **Erection Diagram.**—The erection diagram should show the relative location of every part of the structure; shipping marks for the various members; all main dimensions; number of pieces in a member; packing of pins; size and grip of pins, and any special feature or information that may assist the erector in the field. The approximate weight of heavy pieces will materially assist the erector in designing his falsework and derricks.

6. **Falsework Plans.**—For ordinary structures it is not common to prepare falsework plans in the office, this important detail being left to the erector in the field. For difficult or important work erection plans should be worked out in the office, and should show in detail all members and connections of the falsework, and also give instructions for the successive steps in carrying out the work. Falsework plans are especially important for concrete and masonry arches and other concrete structures, and for forms for all walls, piers, etc. Detail plans of travelers, derricks, etc., should also be furnished the erector.

7. **Bills of Material.**—Complete bills of material showing the different parts of the structure with its mark, and the shipping weight should be prepared. This is necessary in checking up the material to see that it has all been shipped or received, and to check the shipping weight.

8. **Rivet List.**—The rivet list should show the dimensions and number of all field rivets, field bolts, spikes, etc., used in the erection of the structure.

9. **List of Drawings.**—A list should be made showing the contents of all drawings belonging to the structure.

STRUCTURAL DRAWINGS.

METHODS.—The drawings for structural steel work differ from the drawings for machinery in that (*a*) two scales are used, one for the length of the member or the skeleton of the structure, and one for the details; (*b*) members are commonly shown by one projection; and (*c*) the drawings are not to exact scale, all distances being governed by figures.

Two methods are used in making shop drawings.

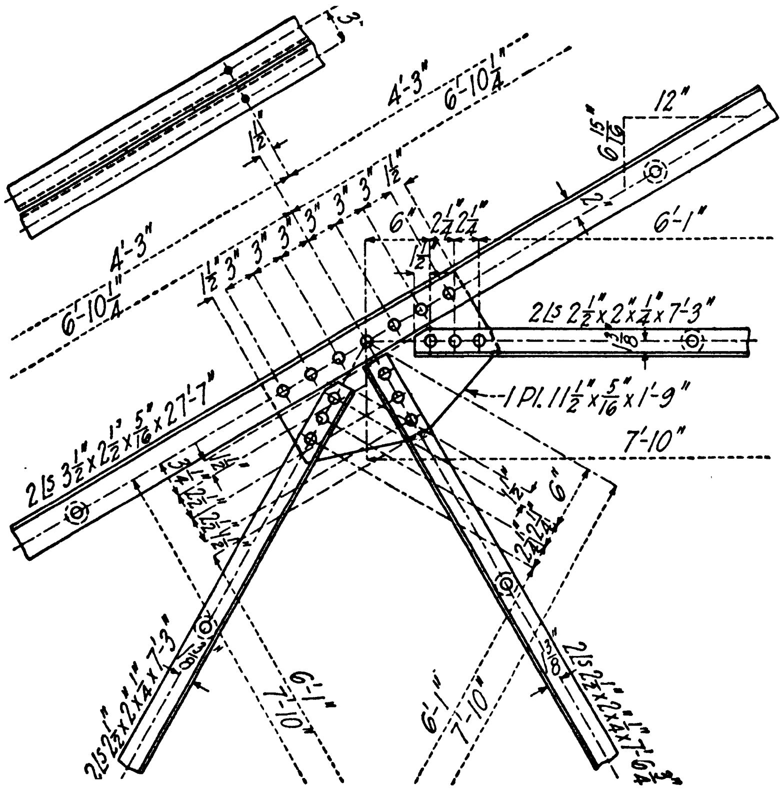


FIG. 1. TRUSS JOINT, COMPLETELY DETAILED.

(1) The first method is to make the drawings so complete that the templets can be made for each individual piece on the bench. This method is used for all large trusses and members, and where there is not room to lay the member out on the templet shop floor. The details for the joint of a Fink roof truss completely detailed are shown in Fig. 1. A joint of a roof truss of the locomotive shop of the A. T. & S. F. Ry., at Topeka, Kansas, is completely detailed in Fig. 2.

(2) The second method is to give on the drawings only sufficient dimensions to locate the position of each member, the number of rivets, and the sizes of members, leaving the details to be worked out by the templet maker on the laying-out floor. Sufficient data should be given to definitely locate the main laying-out points. The interior pieces should be located by center lines corresponding to the gage lines of the angles, or center line of the piece, as the case may be. The rivet spacing should be given complete for members detailed on different sheets, or where it is necessary to obtain a required clearance, and other places where it will materially assist the

templet maker. The drawings should indicate the number and arrangement of the rivets in each connection, as well as the maximum, the usual and the minimum rivet pitch allowed. Sketch details of the joint which was completely detailed in Fig. 1 are shown in Fig. 3, and the outline details of a roof truss by the second method are shown in Fig. 4.

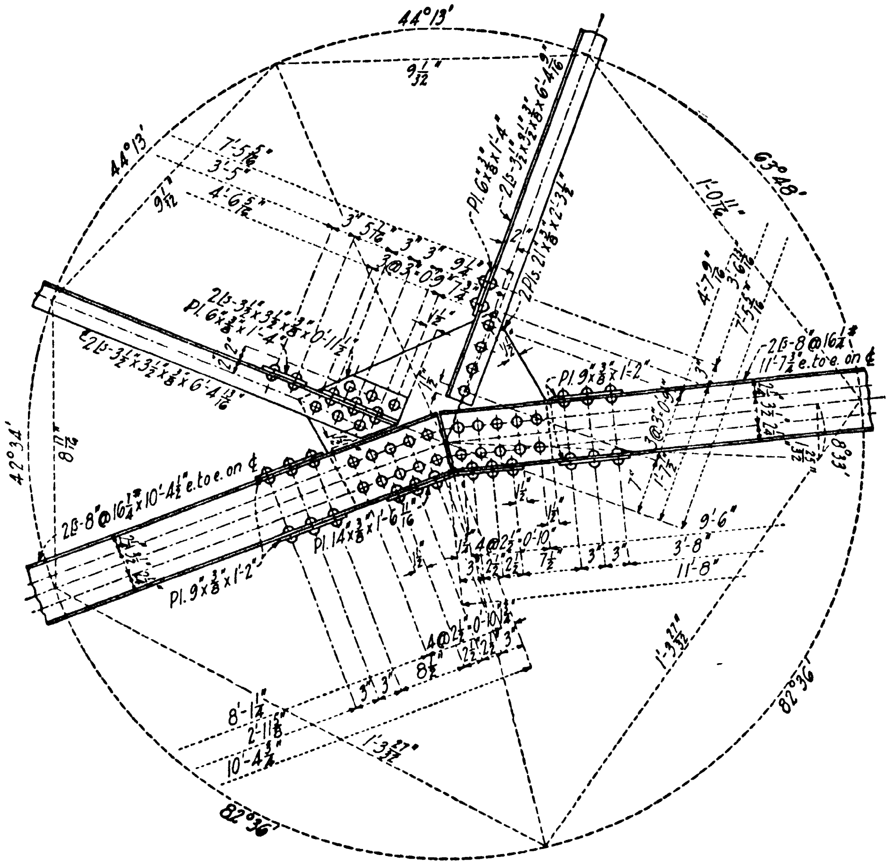


FIG. 2. JOINT OF ROOF TRUSS COMPLETELY DETAILED.
(Section of Shop Details of Roof Truss.)

Members may be detailed in the position which they are to occupy, or they may be detailed separately. For riveted trusses and riveted members the entire truss or member should be detailed in position. The detail shop plans for a riveted brace are shown in Fig. 5. The field rivets are shown by black and the shop rivets by open circles. The center lines are indicated by dotted lines. Light full black lines are commonly used for dimension lines, while red dimension lines are sometimes used but do not make as good blue prints as black lines.

RULES FOR SHOP DRAWINGS.—The following rules are essentially those in use by the best bridge and structural shops.

Size of Sheet.—The standard size of sheet shall be 24 × 36 in. with two border lines ½ and 1 in. from the edge respectively, see Fig. 6. Sheets 18 × 24 in. with two border-lines ½ and 1 in.

from the edge respectively, may also be used. For beam sheets, bills of material, etc., use letter size sheets $8\frac{1}{2} \times 11$ in.

Title.—The title shall be arranged uniformly for each contract and shall be placed in the lower right hand corner. The title shall contain the name of the job, the description of the details on the sheet, the number of the sheet, spaces for approval and other information as shown in Fig. 6.

Scale.—The scale of the lengths of the members or skeleton of the structure shall be $\frac{1}{4}$, or $\frac{3}{8}$, or $\frac{1}{2}$ in. to 1 ft., depending upon the available space and the complexity of the member or structure. Shop details shall as a rule be made $\frac{3}{4}$ or 1 in. to 1 ft. For small details $1\frac{1}{2}$ and 3 in. to 1 ft. may be used; while for large plate girders $\frac{1}{2}$ or $\frac{3}{8}$ in. to 1 ft. may be used.

Views Shown.—Drawings shall be neatly and carefully made to scale. Members shall be detailed in the position which they will occupy in the structure; horizontal members being shown lengthwise, and vertical members crosswise on the sheet. Inclined members (and vertical members

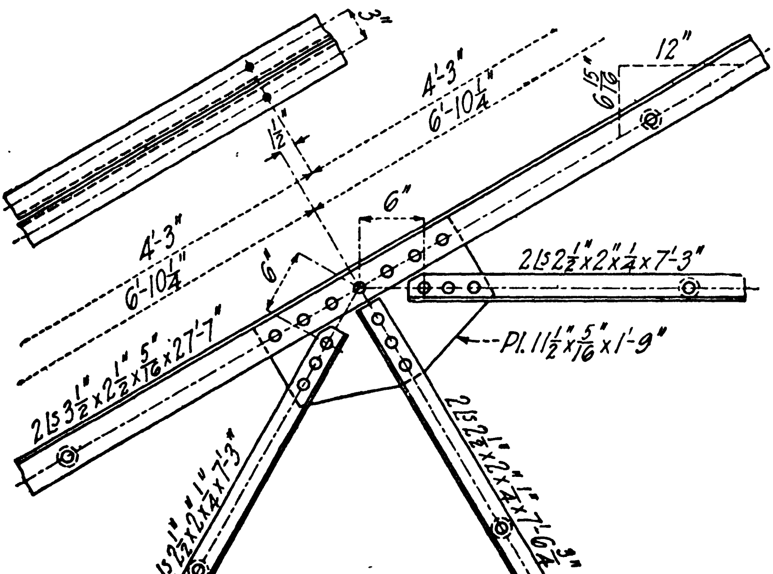


FIG. 3. TRUSS JOINT, SKETCH DETAILED.

when necessary on account of space) may be shown lengthwise on the sheet, but then only with the lower end on the left. Avoid notes as far as possible; where there is the least chance for ambiguity, make another view.

In truss and girder spans, draw the inside view of the far truss, left hand end, Fig. 7. The piece thus shown will be the right hand, and need not be marked right. In cases where it is necessary to show the left hand of a piece, mark "left-hand shown" alongside the shipping mark.

Show all elevations, sections and views in their proper position, looking toward the member. Place the top view directly above, and the bottom view directly below the elevation. The bottom view should always consist of a horizontal section as seen from above.

In sectional views, the web (or gusset plate) shall always be blackened; angles, fillers, etc., may be blackened or cross-hatched, but only when necessary on account of clearness. In a plate

girder, for example, it is not necessary to blacken or cross-hatch all the fillers and stiffeners in the bottom view.

Holes for field connections shall always be blackened, and shall, as a rule, be shown in all elevations and sectional views. Rivet heads shall be shown only where necessary; for example, at the ends of members, around field connections, when countersunk, flattened, etc. In detailing members which adjoin or connect to others in the structure, part of the latter shall be shown in

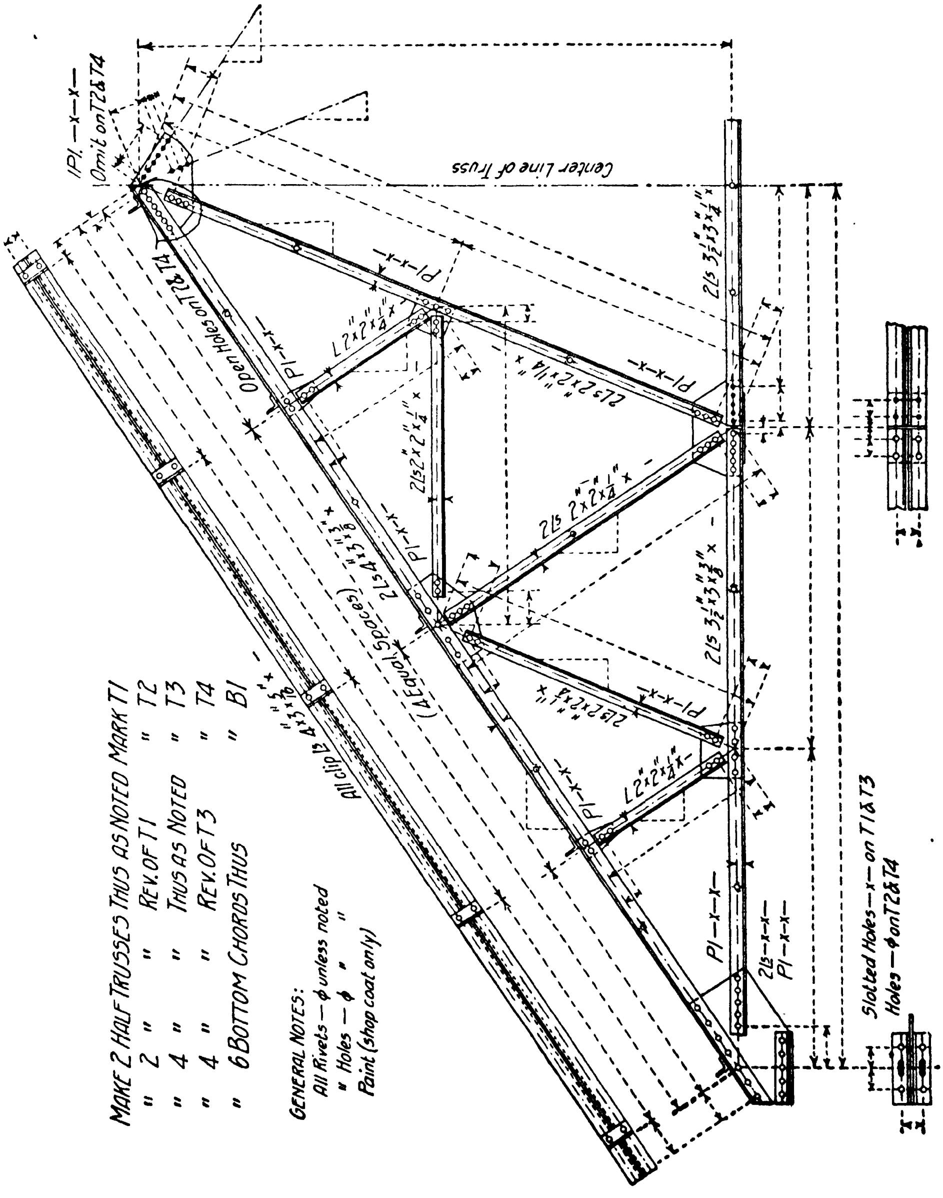


FIG. 4. TRUSS, SKETCH DETAILED.

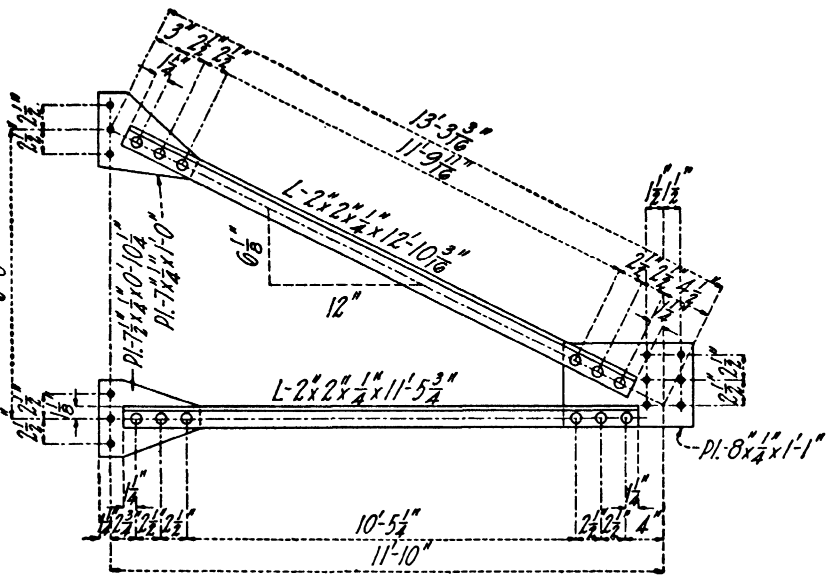


FIG. 5. SHOP DETAILS OF BRACE.

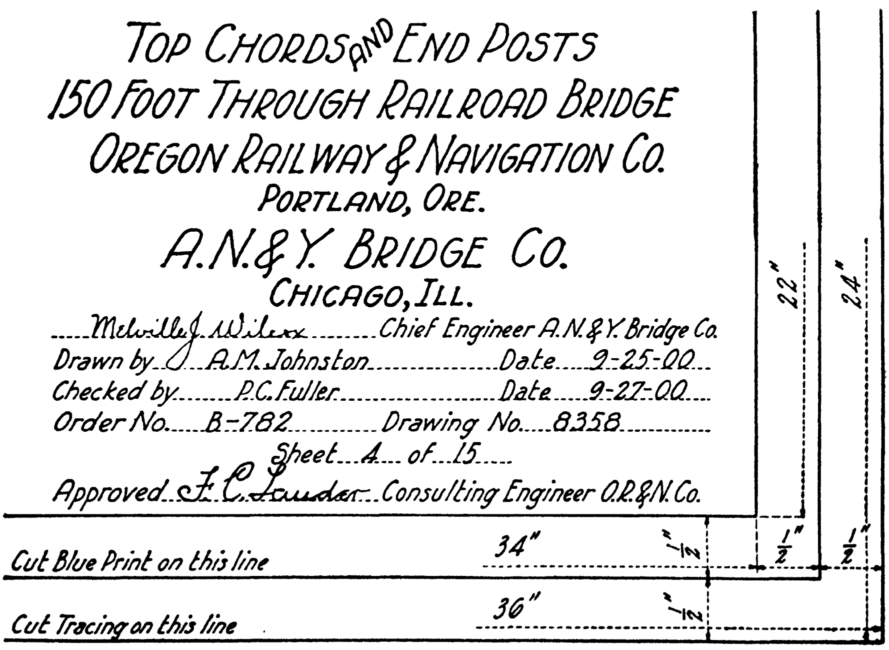


FIG. 6. STANDARD SHEET AND TITLE FOR STRUCTURAL DRAWINGS.

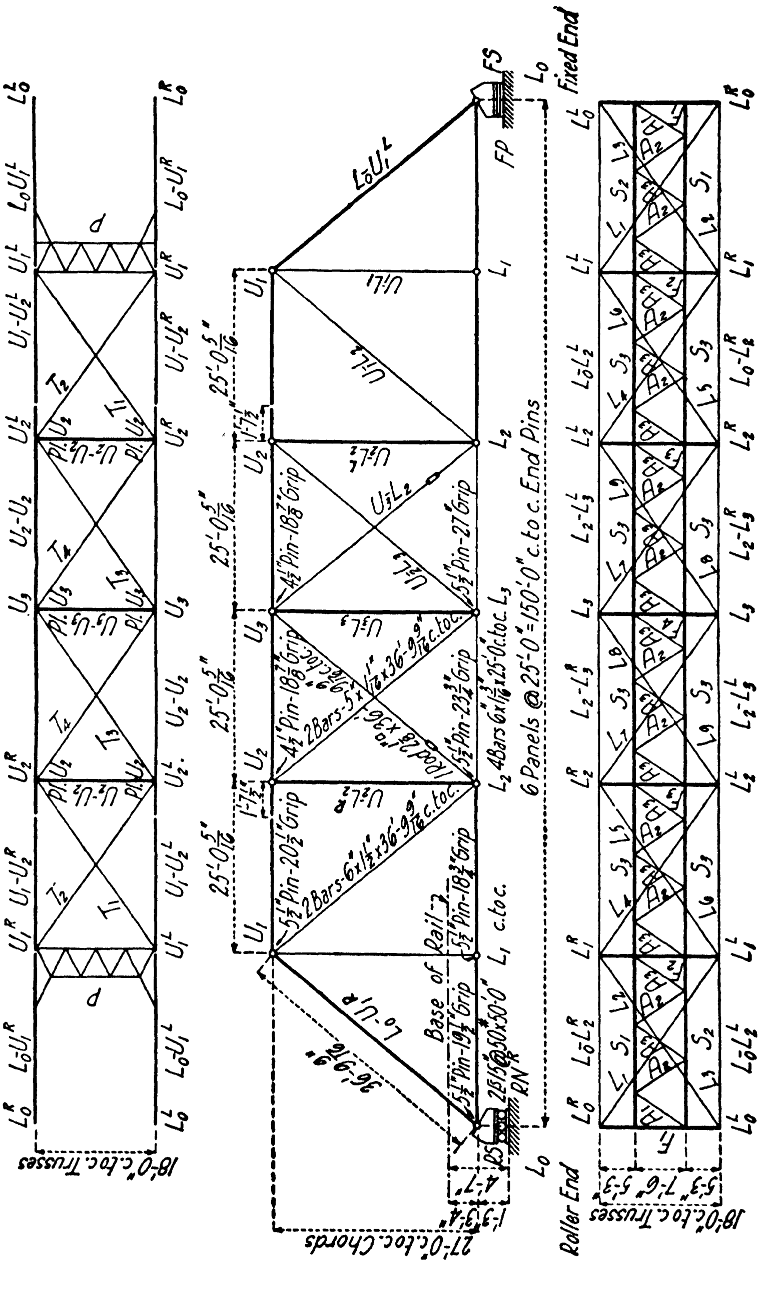


FIG. 7. STANDARD MARKING AND ERECTION DIAGRAM FOR A TRUSS BRIDGE.

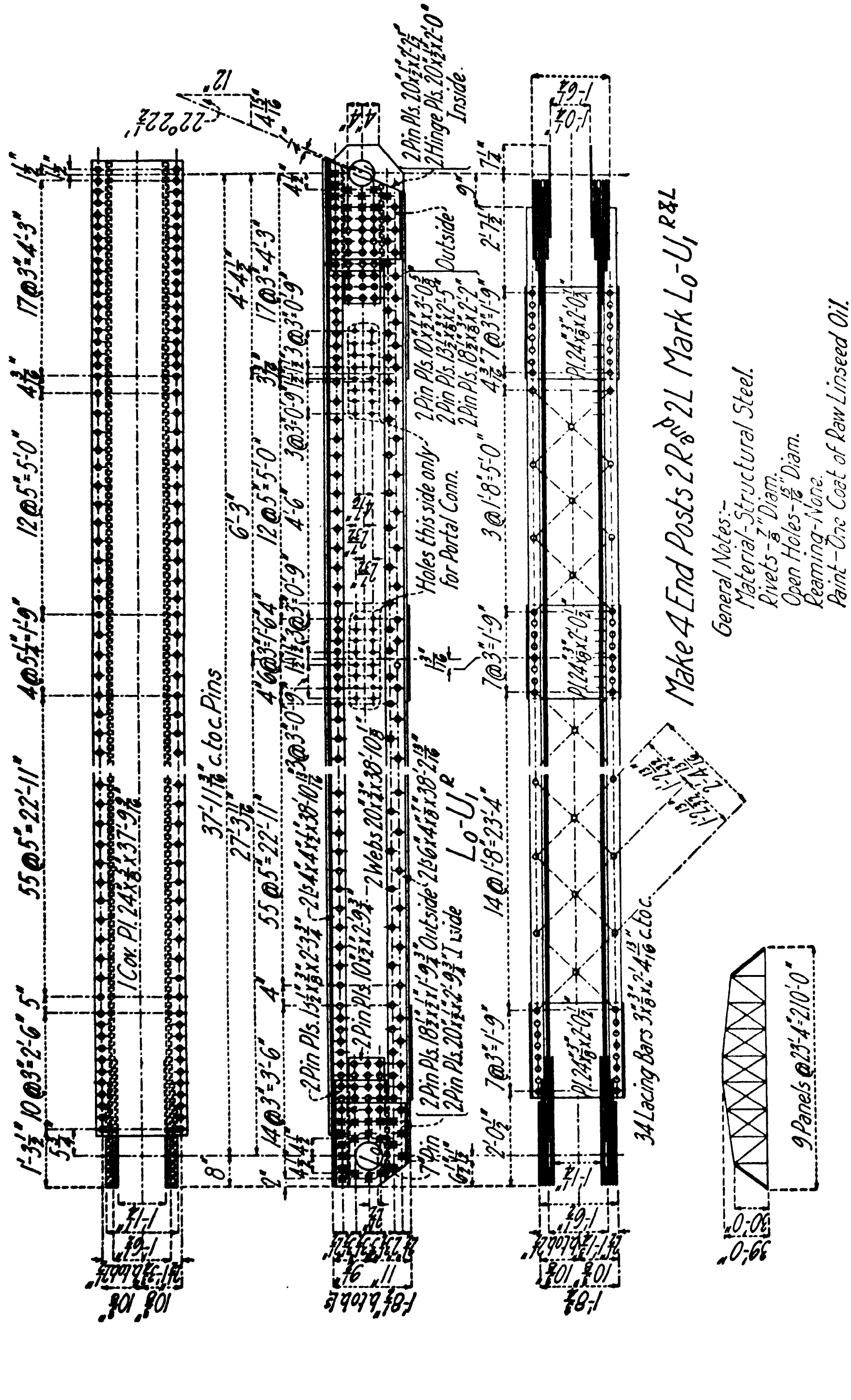


FIG. 8. SHOP DETAILS OF END-POST OF A TRUSS BRIDGE.

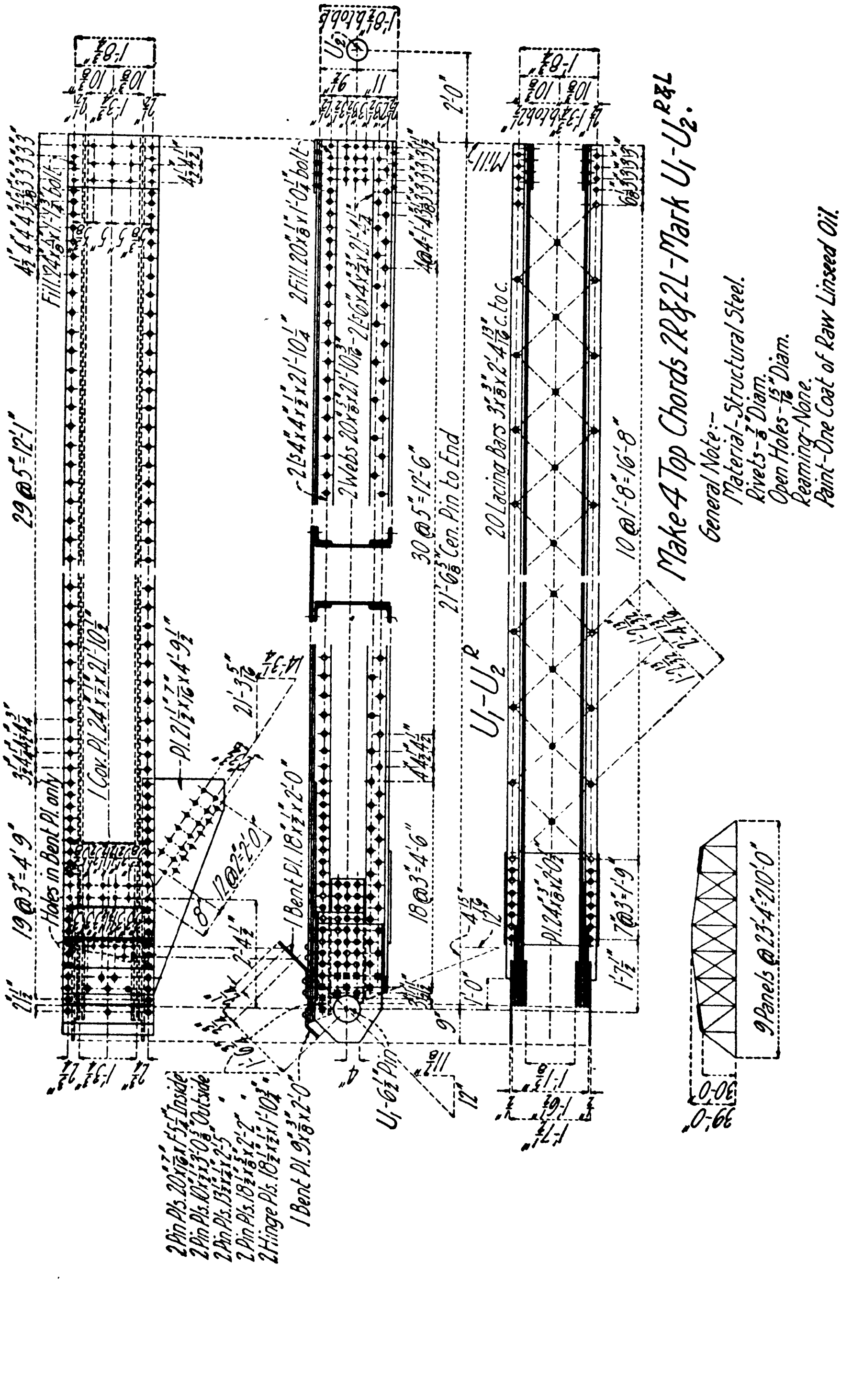


Fig. 9. Shop Details of Top Chord of a Truss Bridge.

dotted lines, or in red, sufficiently to indicate the clearance required or the nature of the connection. Plain building work is exempted from this rule.

A diagram to a small scale, showing the relative position of the member in the structure, shall appear on every sheet, Fig. 8 and Fig. 9. The members detailed on the sheet shall be shown by heavy black lines, the remainder of the structure in light black lines. Plain building work is exempt from this rule.

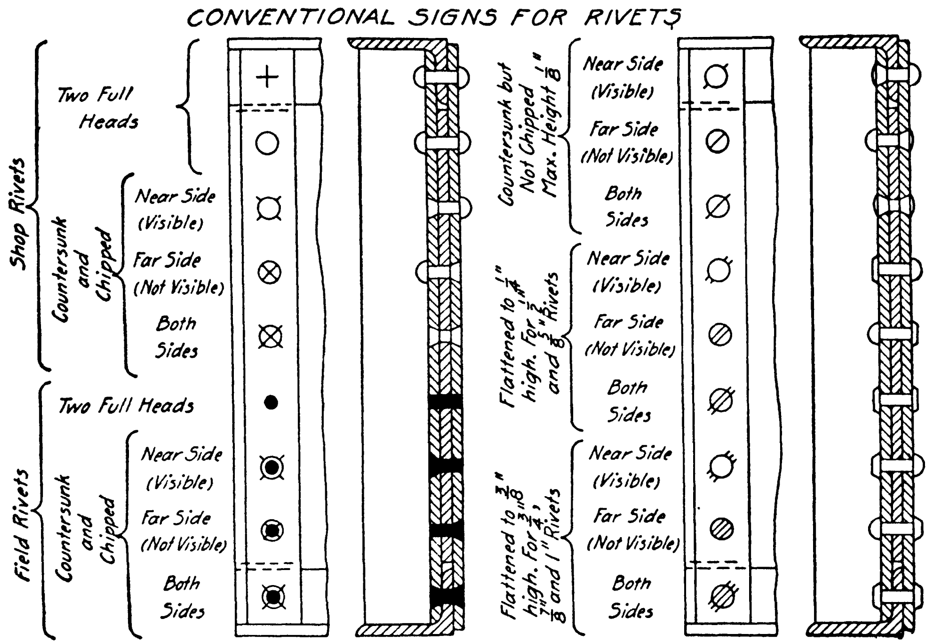


FIG. 10. CONVENTIONAL SIGNS FOR RIVETS.

When part of one member is detailed the same as another member, figures for rivet spacing need not be repeated; refer to previous sheet or sheets, bearing in mind that these must contain final information. *It is not permissible to refer to a sheet, which in turn refers to another sheet.* The section, finished length, and the assembling mark for each member shall be shown on every sheet. Main dimensions which are necessary for checking, such as c. to c. distances, story heights, etc., shall be repeated from sheet to sheet. Holes for field connections must always be located independently, even if figured in connection with shop rivets; they shall be repeated from sheet to sheet unless they are standard, in which case they shall be identified by a mark and the sheet given on which they are detailed.

The quality of material, workmanship, size of rivets, etc., shall be specified on every sheet as far as it refers to the sheet itself. Standard workmanship need not be specified on each sheet.

Lettering.—Engineering News lettering as developed by Reinhardt in his book on freehand lettering shall be used on all drawings. Preferably main titles and sub-titles shall be vertical and the remainder of the lettering inclined. The height of letters shall be as follows: Main titles—capitals 15/50 in., small capitals 12/50 in.; sub-titles—capitals, full height lower case letters and numerals 5/20 in., lower case letters 3/20 in.; other lettering—capitals, full height lower case letters and numerals 5/30 in., lower case letters 3/30 in. Where the drawing is crowded the body of the lettering may be 5/40 in. and 3/40 in. respectively. The following pens are recommended: For

titles Leonardt & Co.'s Ball-Pointed No. 516F; for all other lettering Hunt Pen Co.'s extra fine Shot Point, No. 512. No pen finer than Gillott's No. 303 should be used. Light pencil guide lines shall be drawn for all lettering. All tracings shall be made on the dull side of the tracing cloth. Erasures shall be made with soft rubber pencil eraser and a metal shield. Rubber erasers containing sand destroy the surface of the cloth and make it difficult to ink over the erased spot. The use of knives or steel erasers will not be permitted. Tracings shall be cleaned with a very soft rubber eraser, and not with gasolene or benzine, which destroy the finish of the tracing cloth. All lines shall preferably be made with black India ink; full lines to represent members, dash and dot to represent center lines, and dotted lines (or full light black lines) to represent dimension lines. If permitted by the chief draftsman red ink may be used for dimension and center lines. The ends of dimension lines shall, however, always be indicated by arrows made with black ink.

Conventional Signs.—Conventional signs for rivets are shown in Fig. 10. Countersunk rivets project $\frac{1}{8}$ in.; if less height of rivets is required, drawings shall specify that they are to be chipped, or the maximum projection may be specified. Flattened heads project $\frac{3}{8}$ in. to $\frac{7}{16}$ in.; if less height of heads is required, they shall be countersunk. Metals in section shall be shown as in Fig. 11. Standards for rivets and riveting are given in Part II, which see.

Marking System.—A shipping mark shall be given to each member in the structure, and no dissimilar pieces shall have the same mark. The marks shall consist of capital letters and numerals, or numerals only; no small letters shall be used except when sub-marking becomes absolutely necessary. The letters R and L shall be used only to designate "right" and "left." Never use the work "marked" in abbreviated form in front of the letter, for example say, 3 Floorbeams G4, and not, 3 Floorbeams, Mk. G4. Whenever a structure is divided up into different contracts care should be taken not to duplicate shipping marks. Pieces which are to be shipped bolted on a

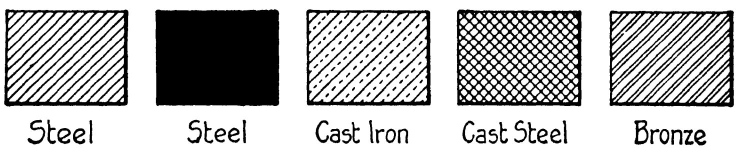


FIG. 11. CONVENTIONAL SIGNS FOR METALS.

member shall also have a separate mark, in order to identify them should they for some reason or another become detached from the main member. The plans shall specify which pieces are to be bolted on for shipment, and the necessary bolts shall be billed. For standard marking system for a truss bridge, see Fig. 7.

A system of assembling marks shall be established for all small pieces in a structure which repeat themselves in great numbers. These marks shall consist of small letters and numerals or numerals only; no capital letters shall be used; avoid prime and sub-marks, such as M_a'. Pieces that have the same assembling mark must be alike in every respect; same section, length, cutting and punching, etc.

Shop Bills.—Shop bills shall be written on special forms provided for the purpose. When the bills appear on the drawings as well, they shall either be placed close to the member to which they belong or on the right hand side of the sheet. When the drawings do not contain any shop bills, these shall be so written that each sheet can have its bill attached to it if desired; one page of shop bills shall not contain bills for two sheets of drawings. In large structures which are subdivided into shipments of suitable size, both mill and shop bills must be written separately for each shipment. In writing the shop bill bear in mind that it shall serve as a guide for the laying out and assembling of the member, besides being a list of the material required. For this reason members which are radically different as to material shall not be bunched in the same shop bill, neither shall pieces which have different marks be bunched in the same item, even if the material

is the same. Bill first the main material in the member, and follow with the smaller pieces, beginning at the left end of a girder, or at the bottom of a post or girder. On a column each different bracket shall be billed complete by itself. Do not bill first all the angles and then all the flats; for example when the end stiffeners in a girder are billed, the fillers belonging to them shall follow immediately after the angles, and so on.

When machine-finished surfaces are required, the drawing and the shop bill shall specify the finished width and length of the piece, the proper allowance for shearing and planing being made in the mill bill. When the metal is to be planed as to thickness, the drawing and the shop bill shall specify both the ordered and the finished thickness; one pl. 15 in. × ¾ in. × 1 ft. 6 in. (planed from 13/16 in.).

Field Rivets.—A "Bill of Field Rivets" shall be made for each structure. The "Bill of Field Rivets" shall give in order the number, diameter, grip, length and the location of the rivets in the structure. The number of field rivets to be furnished to the erector shall be the actual number of each diameter and length required, plus 15 per cent, plus 10.

Field bolts shall be billed on "bill of rivets and bolts" only. Bill them similarly to field rivets, and give the drawing number on which they are shown; 4—bolts ⅞ in. × 2 in. grip, 3 in. U. H. stringers "S" to floorbeam "F" drawing No. 13, 4 hex. (or 4 square) nuts for above bolts. Bill of bolts and bill of field rivets shall be prepared and placed in the shop in time to be made with other material.

General Notes.—Full information regarding the following points shall appear on the drawings, where practicable as "General Notes." Loading ...., Specifications ...., Material ...., Rivets ...., Open Holes ...., Reaming Requirements ...., Other Special Requirements ...., Painting.

Erection Plan.—Make erection plans simultaneously with the shop plans, and keep same up to date. The erection plans must show plainly the style of connections; joints in pin spans are to be shown separately to a larger scale. For the erection plan of a truss bridge see Fig. 7. Shipping bills showing the number of pieces, erection mark, and weight shall be made for each shipment.

Subdivisions.—Every contract embracing different classes of work shall have a subdivision for each class. These subdivisions will be furnished by the chief draftsman. Drawings, shop and shipping bills must be kept separate for each class.

PLATE GIRDER BRIDGES.—General Rules.—The plate girder span shall be laid out with regard to the location of web splices, stiffeners, cover plates, and in a through span, floorbeams and stringers, so that the material can be ordered at once. Locate splices and stiffeners with a view of keeping the rivet spacing as regular as possible; put small fractions at the end of girder. Stiffeners, to which cross-frames or floorbeams connect, must not be crimped, but shall always have fillers. The outstanding leg shall not be less than 4 in., gaged 2¼ in.; this will enable cross-frames or floorbeams to be swung into place without spreading the girders. The second pair of stiffeners at the end of girder over the bed-plate shall be placed so that the plate will project not less than 1 in. beyond the stiffeners.

Always endeavor to use as few sizes as possible for stiffeners, connection plates, etc., and avoid all unnecessary cutting of plates and angles. For this purpose locate end holes for laterals and diagonals so that the members can be sheared in a single operation. In spans on a grade, unless otherwise specified, put the necessary bevel in the bed-plate and not in the base-plate. In short spans, say up to 50 ft. put slotted holes for anchor-bolts in both ends of girders, ⅜ in. larger diameter than the anchor bolts.

In square spans, show only one-half, but give all main dimensions for the whole span. In skew spans show the whole span; when the panels in one-half of span are same as in the other half, give the lengths of these panels, but do not repeat rivet-spacing, except where it differs.

In the small scale diagram, which shall appear on every sheet, unless span is drawn in full, show the position of stiffeners, particularly those to which cross-frames or floorbeams connect.

Deck Plate Girder Spans.—On top of sheet show a top view of span, with cross-frames, laterals and their connections complete, with the girders placed at right distances apart. Below

this view show the elevation of the far girder as seen from the inside, with all field holes in flanges and stiffeners indicated and blackened. At one end of the elevation show in red the bridge-seat and back wall, give figures for distance from base of rail to top of masonry, notch of ties, depth of girder, thickness of base-plate and of bed-plate or shoe. When the other end of girder has a different height from base of rail to masonry, give both figures at the one end, and specify "for this end" and "for other end." If span has bottom lateral bracing, a bottom view (horizontal section) shall be shown below the elevation. When no bottom laterals are required, show only end or ends of lower flange of girder, giving detail of base-plate and its connection to the flange. Detail the bed-plate separately, never show it in connection with the base-plate.

Cross-frames shall, whenever possible, be detailed on the right hand of the sheet in line with the elevation. The frame shall be made of such depth as to permit it being swung into place without interfering with the heads of the flange rivets in the girders. Always use a plate, not a washer with one rivet, at the intersection of diagonals. In skew spans it is always preferable to have an uneven number of panels in the lateral system.

Through Plate Girder Spans.—Show on top of sheet an elevation of the far girder as seen from inside; below this view show a horizontal section of span as seen from above with the lateral system detailed complete. It is generally best to show floorbeams and stringers in red in this view and to detail them on a separate sheet. The stiffeners in a through span should always be arranged so that the floor system can be put in place from the center towards the ends. What is said under "deck spans" about showing bridge-seat, back wall, detailing bed-plate separately, etc., applies to through spans as well.

TRUSS BRIDGES.—General Rules.—Before any details are started all c. to c. lengths of chords, posts, diagonals, etc., shall be determined, and sketches made of shoes, panel-points, splices, etc., so that the material can be ordered as soon as required.

If not otherwise specified, camber shall be provided in the top chord by increasing the length $\frac{1}{8}$ in. for every 10 ft. for railroad bridges, and $\frac{3}{16}$ in. for every 10 ft. for highway bridges. This increase in length shall not be considered in figuring the length of the diagonals, except in special cases, as directed by the engineer in charge. Half the increase in length shall be considered in figuring the length of the top laterals. Particular attention must be paid to what is said under "General Rules" about showing part of adjoining member in red, and about the small scale diagram on every sheet.

For every truss bridge an erection diagram shall be made on a separate sheet, giving the shipping marks of the different members and all main dimensions, such as c. to c. trusses, height of truss, number and length of panels, length of diagonals, distance from base of rail to masonry, distance from center of bottom chord or pin to masonry, size and grip of pins (Fig. 7), also show in larger scale the packing at panel points, state any special feature which the erector needs to look out for, and give approximate weight of heavy and important pieces when their weight exceeds five tons. If in any place it is doubtful whether rivets can be driven in the field, the erection diagram and also the detail drawings shall state that "turned bolts may be used if rivets cannot be driven." A list giving number and contents of drawings belonging to the bridge shall also appear on the erection diagram sheet.

Riveted Truss Bridges.—In square spans, not too large, show the left half of the far truss as seen from the inside and detail all members in their true position, making scale of the skeleton one-half the scale of the details. In skew spans, not symmetrical, show the whole of the far truss. In large spans detail every member separately. When detailing web members bear in mind that the intersection point on the chord must not be used as a working point for a member which stops outside of the chord. A separate working point, preferably the end rivet, shall be established on the member proper, and shall be tied up with the intersection point on the chord.

The clearance between the chord and a web member entering same shall, whenever possible, be not less than $\frac{1}{4}$ in. in heavy and $\frac{1}{16}$ in. in light structures.

Members shall be marked with the panel points between which they go, for example, end-post L_0-U_1; hip vertical U_1-L_1; top chord U_1-U_2, etc., see Fig. 7.

Pin-connected Truss Bridges.—In pin-connected truss bridges detail the left half of the far truss as seen from the inside, every member by itself. It is generally best to commence with the end-post, showing it lengthwise on the sheet with the lower end to the left; then the first section of the top chord, and so on. The packing at panel points shall, whenever possible, be so arranged that, besides the customary allowance of $\frac{1}{16}$ in. for every bar, a clearance of not less than $\frac{3}{8}$ in. can be provided between the two sides of the chord. When two or more plates are used, $\frac{1}{32}$ in. should in addition be allowed for each plate. Members shall be marked the same as for riveted truss bridges, with the panel points between which they go, see Fig. 7.

Order of Detailing Truss Spans.—In making detail plans and bills of material the following order shall be followed for truss spans.

1. General drawing;
2. End-posts;
3. Upper chords;
4. Lower chords;
5. Intermediate posts;
6. Sway bracing;
7. Upper laterals;
8. Lower laterals;
9. Floorbeams;
10. Stringers;
11. Castings, bolts, eye-bars, pins, etc.

OFFICE BUILDINGS AND STEEL FRAME BUILDINGS.—Number of Drawings.—The different sheets shall be numbered consecutively, whether large or small. No half numbers are permissible except in emergency cases. It is always well to arrange the number so that the sheets follow in the order in which the material is required at the building. The following is generally a good order:

1. Floor plans for all floors;
2. Column schedule;
3. Cast-iron bases for columns;
4. Foundation girders;
5. Foundation beams;
6. First tier of columns;
7. Riveted girders, connecting to first tier of columns
8. Beams connecting to first tier of columns;
9. Miscellaneous material for above;
10. Second tier of columns, etc., etc.

Floor Plans.—Floor plans, Fig. 12, shall, as a rule, be made to a scale $\frac{1}{8}$ in. to 1 ft. A separate plan shall be made for each floor, unless they are exactly alike. Columns shall be marked consecutively with numerals, the word Col. always appearing in front of the numeral, for example, Col. 20. The architect or engineer has generally on his drawing adopted a system of marking for the columns, which should be adhered to, unless altogether too impracticable. Riveted girders shall be indicated with two (2) fine lines when they have cover plates, and with four (4) fine lines when they have no cover plates. They shall be marked consecutively with numerals, using the same marks for girders which are alike. Beams and channels shall be indicated with one single heavy line. They shall be marked the same as girders, with numerals, using same marks when alike. Tie-rods shall be indicated with one single fine line; they need not have any marks. The marking system shall be as uniform as possible for the different floors, i. e., a beam which goes between Col. 2 and Col. 3 shall be marked with the same numeral throughout all the floors. All figures necessary for making the details shall, as a rule, appear on the floor plan, care being taken in writing same to leave room for the erection marks, which must be printed in heavy type above the line or lines representing a beam or girder.

Column Schedule.—For every large building a schedule of the columns shall be made before the details are started, see Fig. 13. Each column, even should several be alike, shall have a separate space, in which shall be given the material and the finished length. As soon as the detail drawings for one tier of columns are finished the sheet numbers shall be inserted as shown on the sample schedule, Fig. 13, making the schedule serve as an index for the column drawings.

Columns.—Columns shall, whenever possible, be drawn standing up on the sheets as they appear in the building. If it becomes necessary to draw them lengthwise on the sheet, the base shall be to the left. Particular attention shall be paid to establishing a marking system for brackets, splice-plates, etc. A summary of all these standard pieces shall be made for each tier

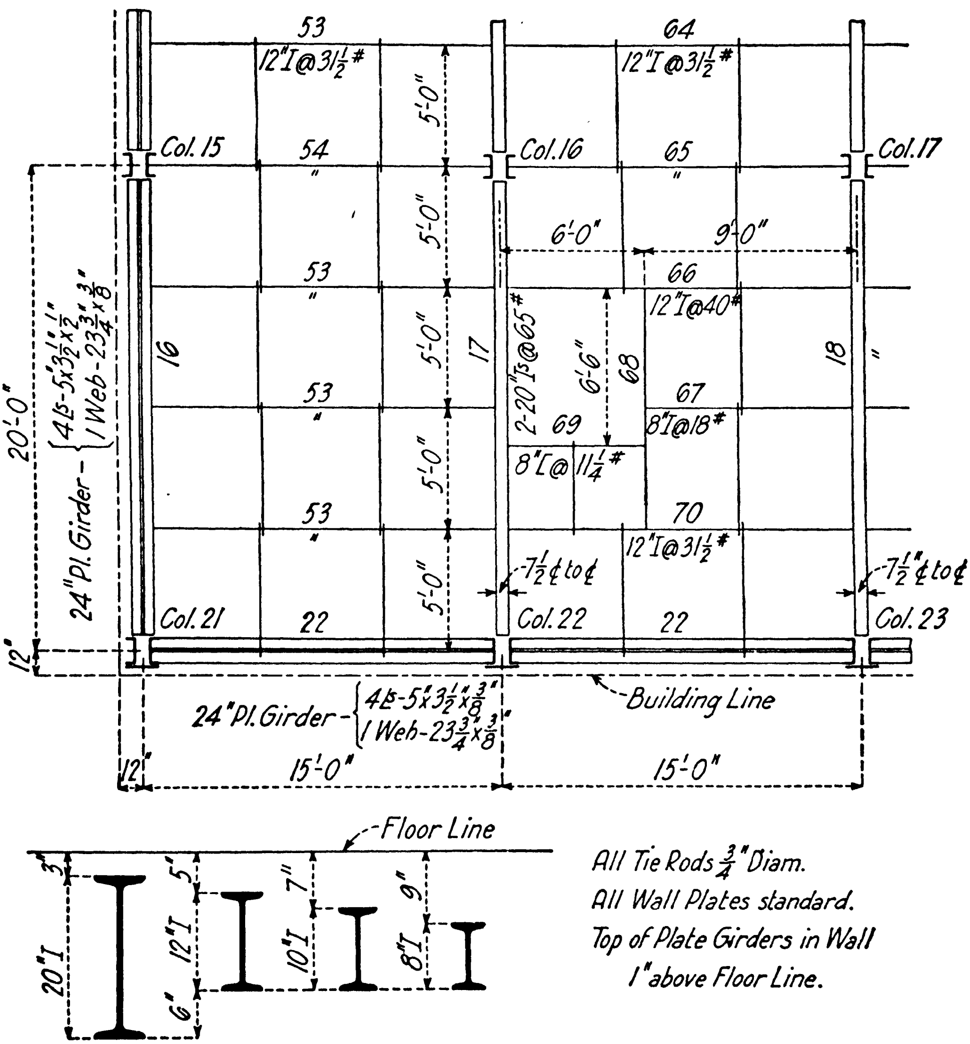


FIG. 12. FLOOR PLANS FOR OFFICE BUILDINGS.

and sent to the shop as early as practicable, in order that they may be gotten out before the main material is taken up. The material for the small pieces shall, as far as possible, be chosen from stock sizes. Columns shall be marked with the numbers of the floors between which they go; Col. 5 (1–3). The lower tier is best marked "Basement Tier." Standard details for columns are given in Fig. 14 and Fig. 15.

Riveted Girders.—Girders shall be marked with the number of the floors, not with letters,

unless requested; for example, 2d Floor, No. 5. What is said under columns about marking system for standard pieces applies to girders as well. When a girder is unsymmetrical about the center line, and a question may arise how to erect it, one end shall be marked with the number of the column to which it connects, or with North, South, East or West. Girders must not be bunched

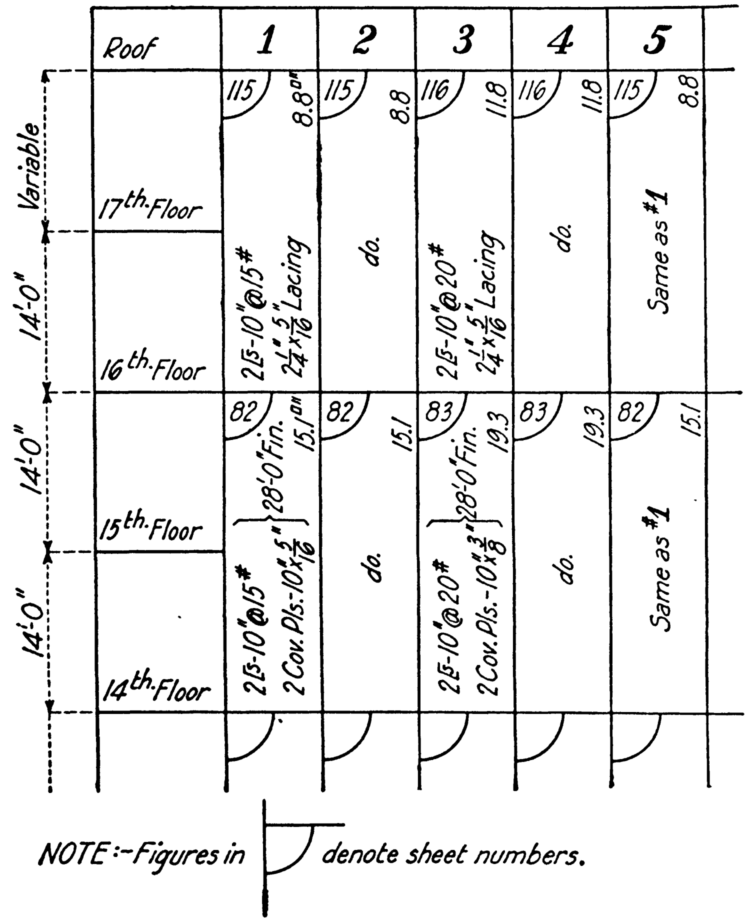


FIG. 13. COLUMN SCHEDULE FOR OFFICE BUILDINGS.

together for the different floors more than to meet the requirements in the field; but they must correspond to the tiers of columns as they will be erected.

Beams.—Beams shall be drawn on the standard forms provided for the purpose. They need not be drawn to scale, see Fig. 16 and Fig. 17. Beams shall be marked the same as girders with the number of the floor; One 12" I @ 40 lb. × 19'-3½", (Mark) 2d Floor No. 35. What is said under girders about marking one end, when not symmetrical around the center line, and about not bunching the different floors more than to meet the requirements in the field, applies to beams as well.

Whenever possible use standard framing angles, Tables 117 and 118, Part II. If it is deemed necessary to use 6 in. × 6 in. angles, punch both legs the same as the 6 in. leg of standard; in 3½ in. × 3½ in. or 4 in. × 3½ in. angles, punch both legs the same as 4 in. leg of standard. It is not abso-

lutely imperative that the gage of the framing angles shall be standard as long as the vertical distance between the holes and in the 6 in. leg the horizontal distance ($2\frac{1}{4}$ in.), are kept standard. Holes for connections, tie-rods, etc., shall be located from one end of the beam, preferably the left. If one end rests on the wall and the other end is framed, then figure from the latter end, be it right

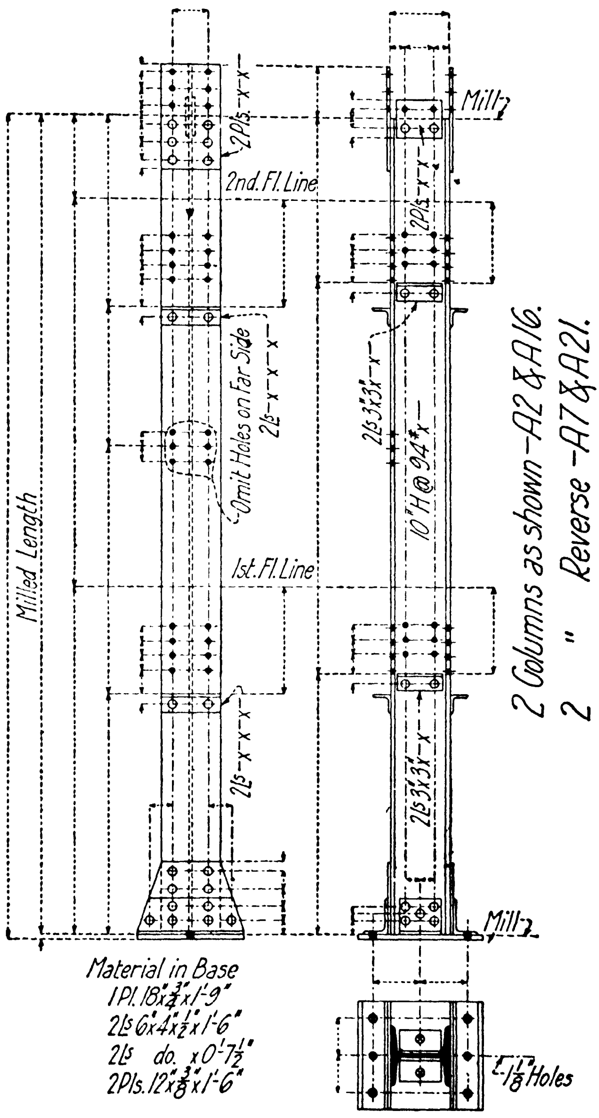


FIG. 14. STANDARD DETAILS FOR BETHLEHEM H-COLUMNS.

or left. This rule may be dispensed with in case of numerous holes regularly spaced in web or flange for connection of shelf-angles, buckle-plates, etc. The allowed overrun at ends of beams must always be indicated, either by giving figures or by showing wall bearing. Holes at the end

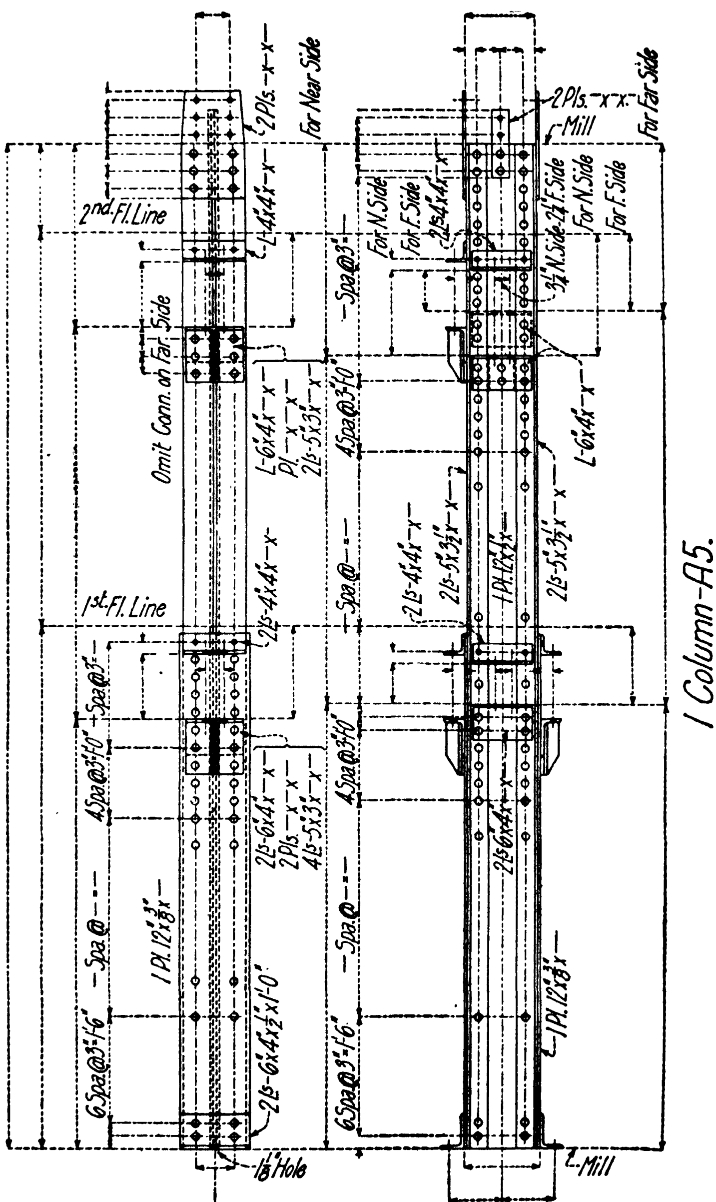


FIG. 15. STANDARD DETAILS FOR BUILT-UP H-COLUMNS.

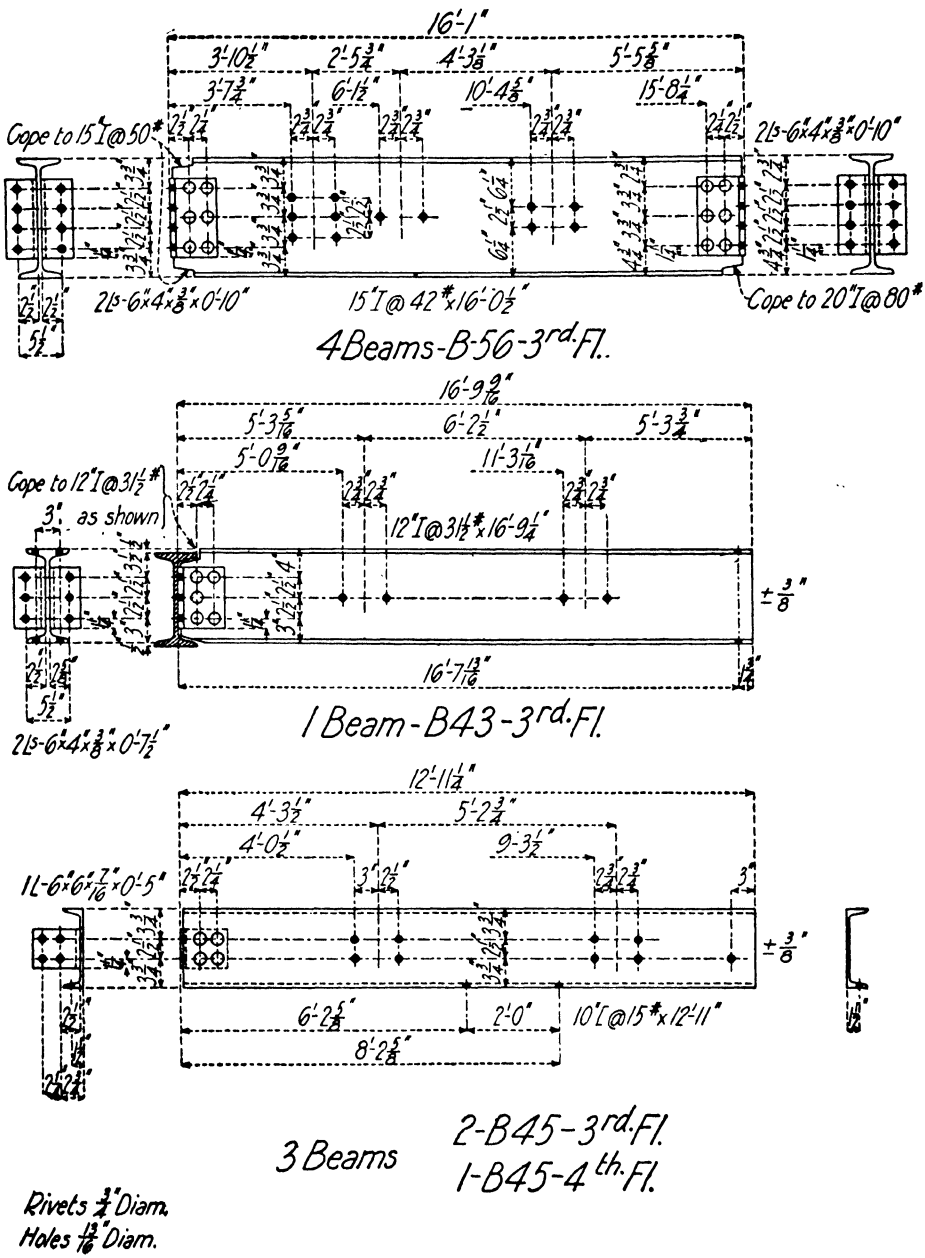


FIG. 16. STANDARD DETAILS FOR ROLLED BEAMS.

of beam for anchors are best figured from wall end, not connecting them with other figures. The distance between end holes in beams which connect through web or flange to columns, girders, etc., shall always be given. When framing angles are standard, do not give any figures for either shop or field rivets, except the distance from bottom of beam to center of connection or to first holes in framing angle, and the horizontal distance between field holes. When special framing angles are used, the fact must be noted and figures given for gages, etc. For standard connection holes in web of beam all figures required are the distance from bottom of beam to centre of connection or to first hole and the horizontal distance between holes. Whenever possible use standard punching.

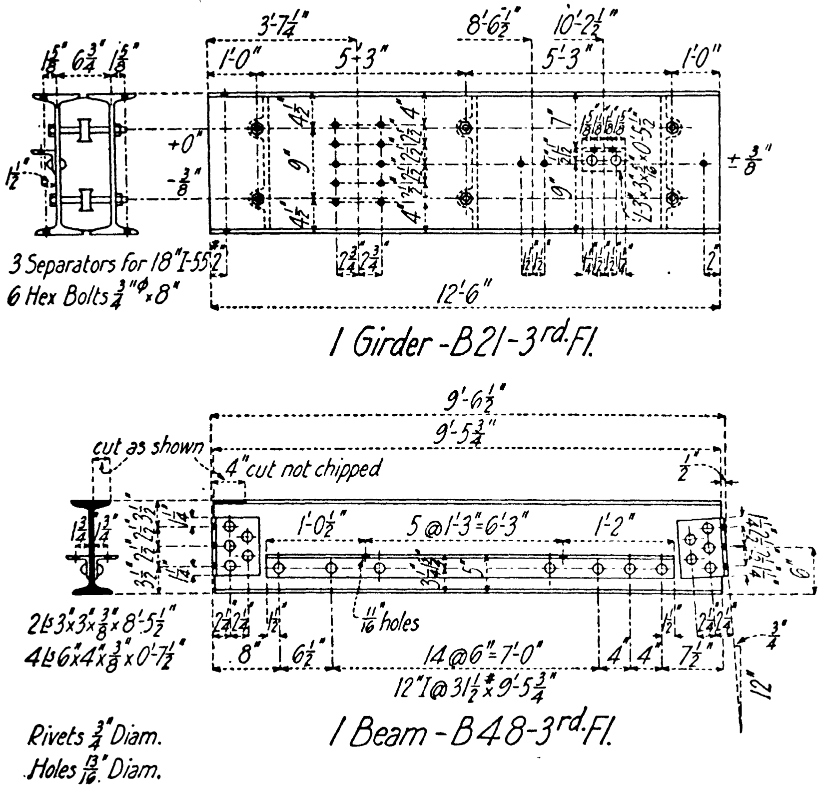


FIG. 17. STANDARD DETAILS FOR ROLLED BEAMS.

Steel Frame Mill Buildings.—The preceding methods will need considerable modification for steel frame mill buildings.

Columns.—Details of steel columns for steel mill buildings are shown in Chapter I and in Fig. 18b and Fig. 18c. The column in Fig. 18b carries one end of a truss with a knee brace and also carries a crane girder. Details of column bases are shown in Fig. 18c.

Plate Girders.—Details of two plate girders designed to carry a moving crane are given in Fig. 18d. The top flanges are made of two angles and a cover plate, while the bottom flanges are made of two angles. Girder G9 is designed to be riveted to a column, while girder G10 is designed to rest on a pedestal and also to carry a second girder which is to be riveted to the end stiffeners. The stiffeners have fillers under them, making crimping unnecessary. The intermediate stiffeners might have been crimped over the flange angles and the filler omitted. Fillers should always be used under the end stiffeners.

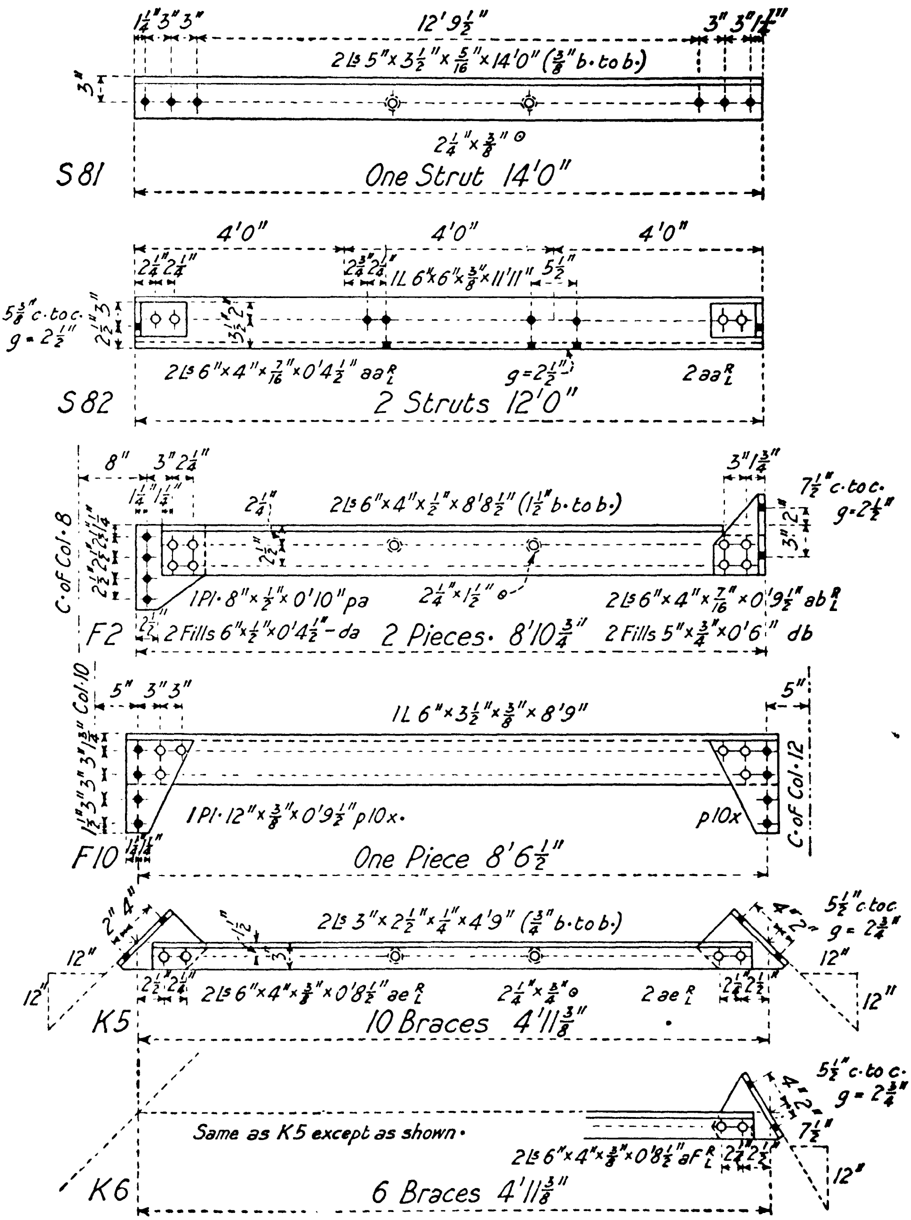


FIG. 18. STANDARD DETAILS FOR ANGLE STRUTS.

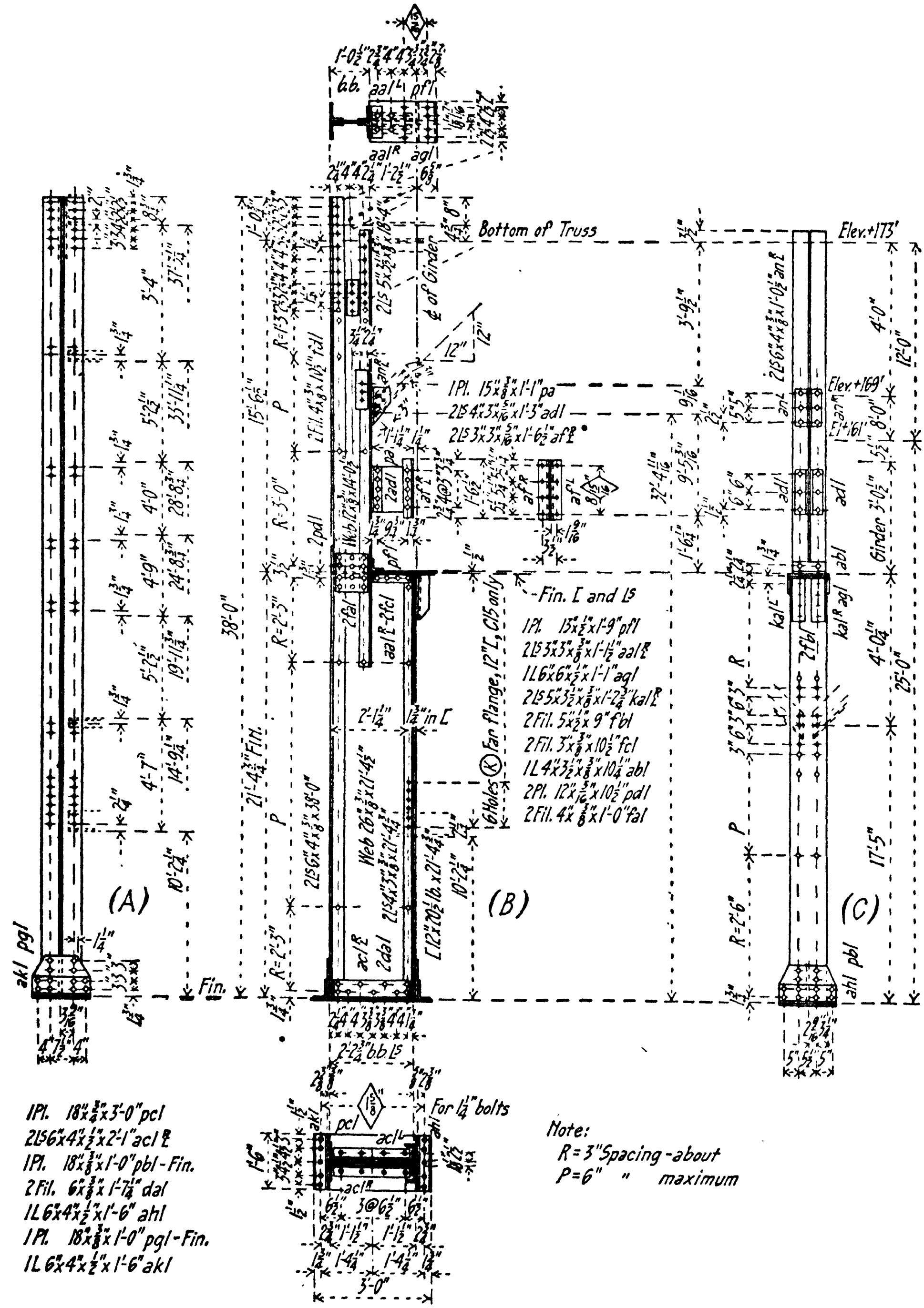


FIG. 18b. DETAILS OF COLUMN. AMERICAN BRIDGE COMPANY.

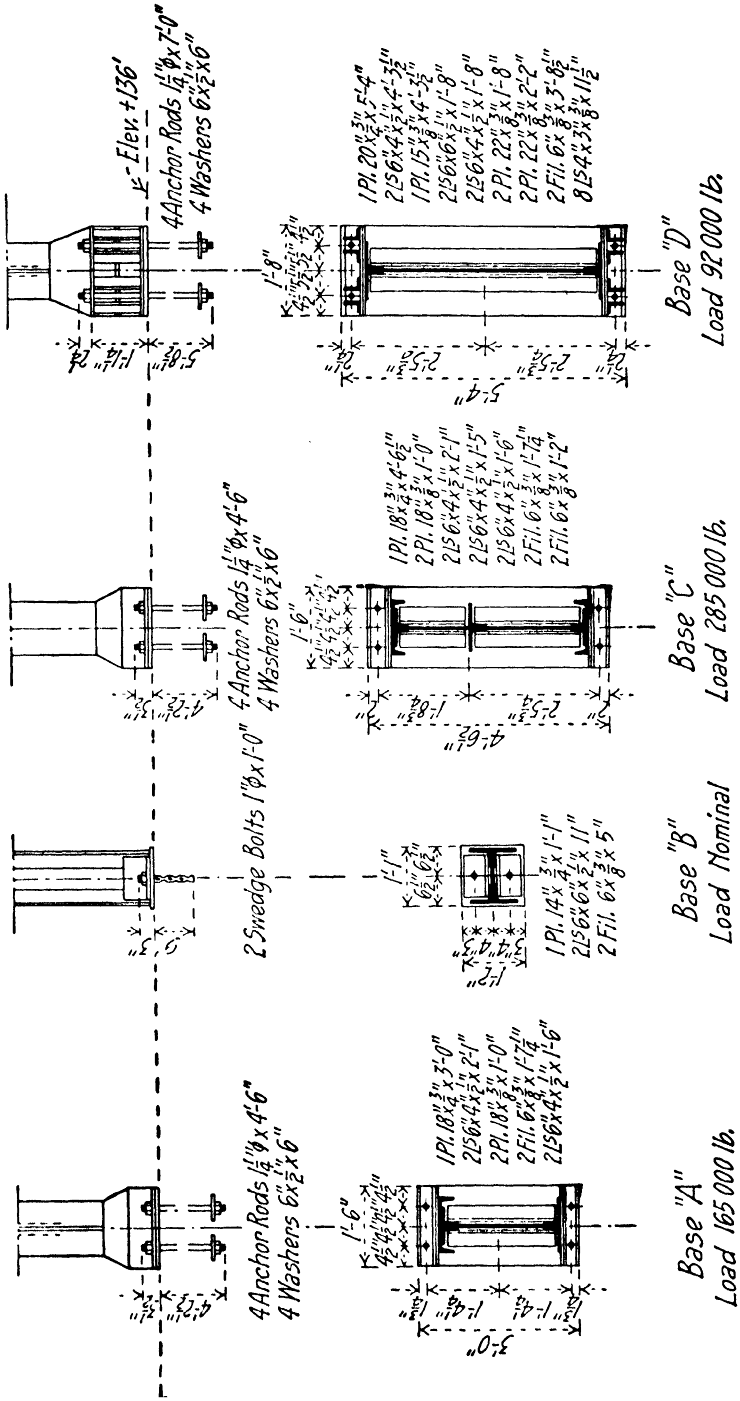


Fig. 18c. Bases for Columns. American Bridge Company.

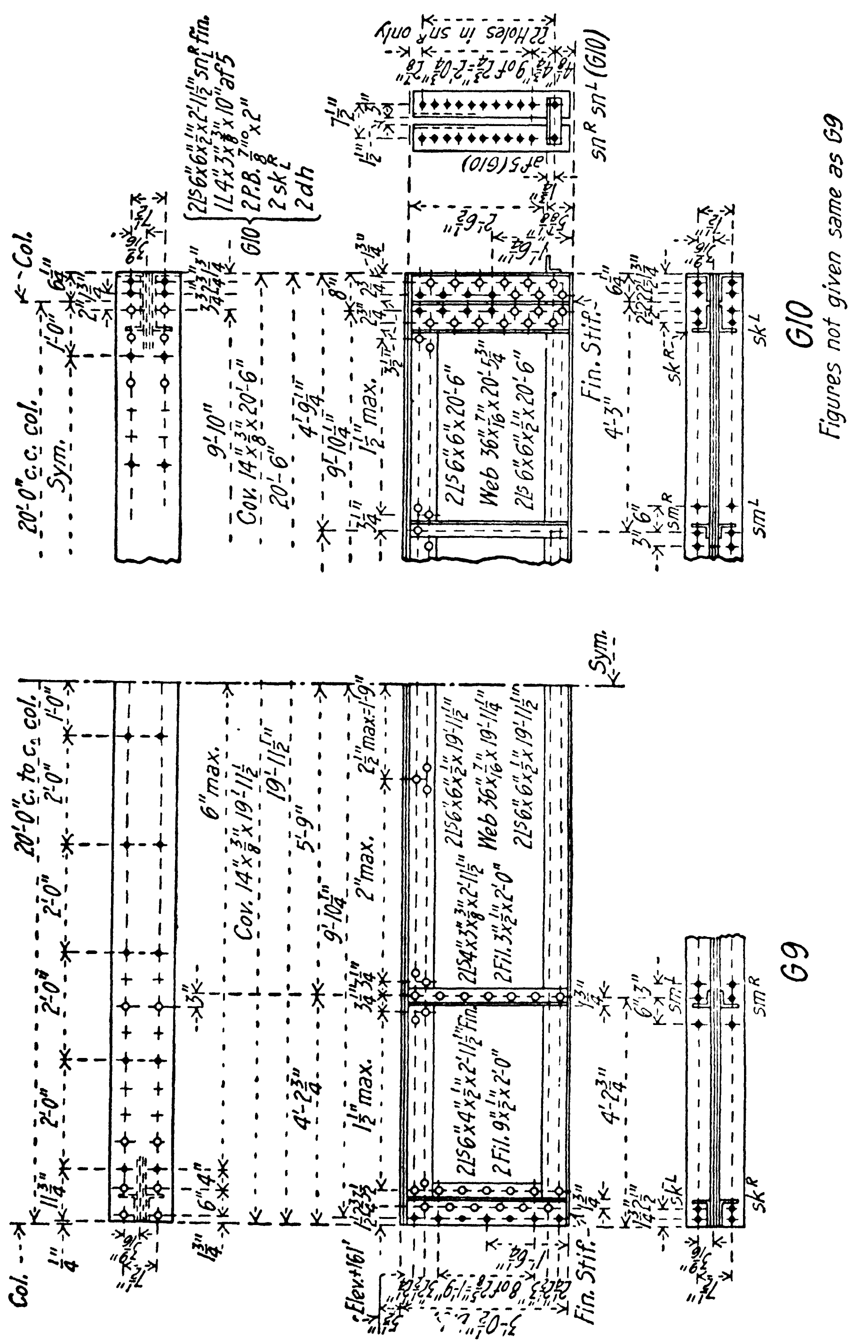


Fig. 18d. Details of Crane Girders. American Bridge Company.

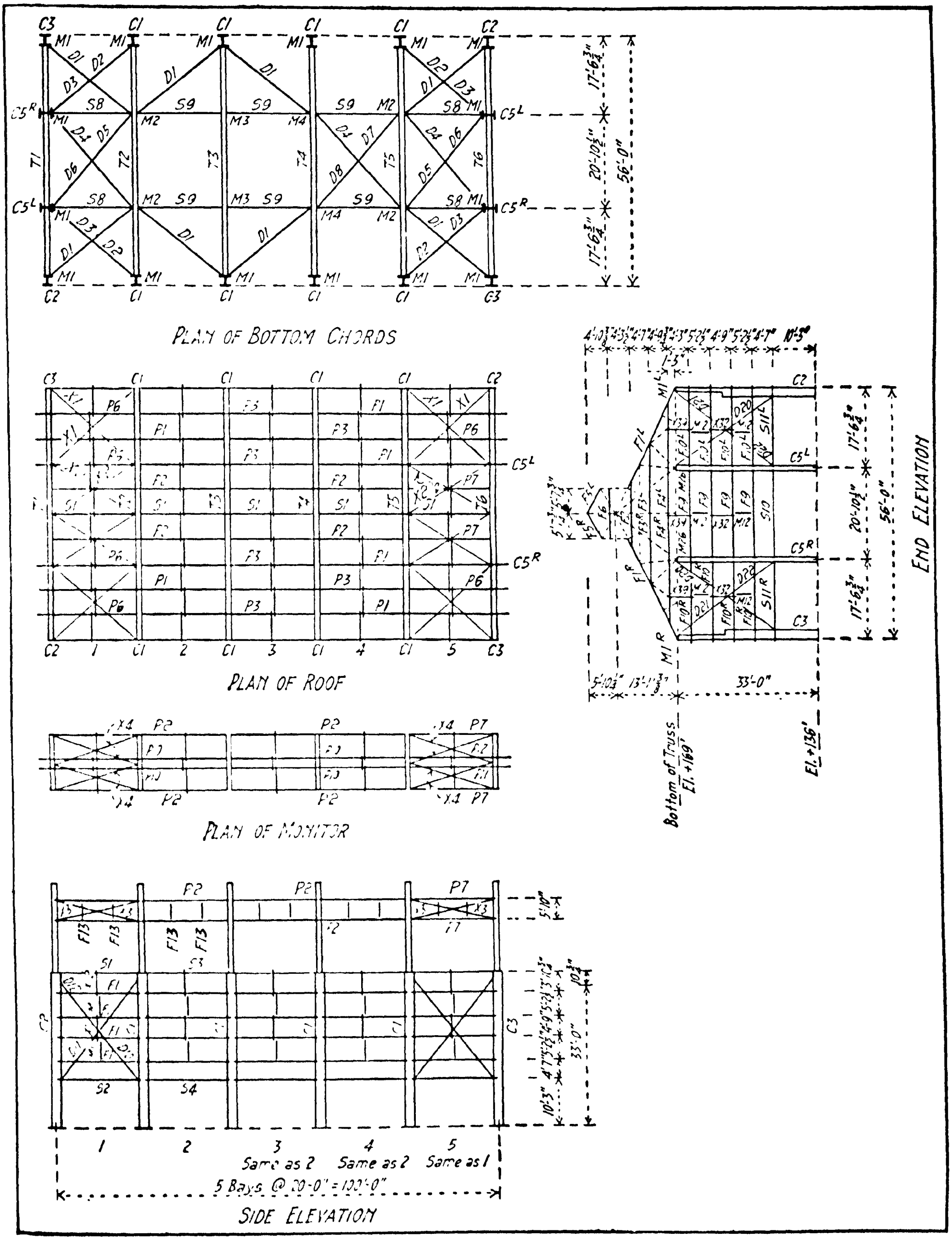


FIG. 18e. ERECTION PLAN OF A STEEL MILL BUILDING.

Erection Plans for Steel Mill Buildings.—The method of making erection plans for steel frame mill buildings shown in Fig. 18e has been found to be very satisfactory.

Where framework is symmetrical about a center line, as trusses and end bents, the members that are reversed should be marked *R* (right) and *L* (left). The right hand side of the end bent is determined by looking outward. Mark trusses T_1, T_2, T_3, etc. Mark posts C_1, C_2, C_3, etc. Mark purlins P_1, P_2, P_3, etc. Mark struts S_1, S_2, S_3, etc. Mark girts F_1, F_2, F_3, etc. Mark bracing D_1, D_2, D_3, etc. The scheme can be modified to suit special conditions.

A foundation plan showing the piers and location of the anchor bolts should be prepared in addition to the erection plan in Fig. 18e.

DETAIL NOTES.—Sections.—End views of sections shall be shown as in (a) Fig. 19, and sections shall be cross-hatched or blackened as shown in (b) Fig. 19.

Assembling Note.—Covers, webs, flange angles, etc., must not be marked alike when it would be necessary to turn them end for end, see (c) Fig. 19.

Rivet Spacing.—Rivet spacing must be tied up from end to end.

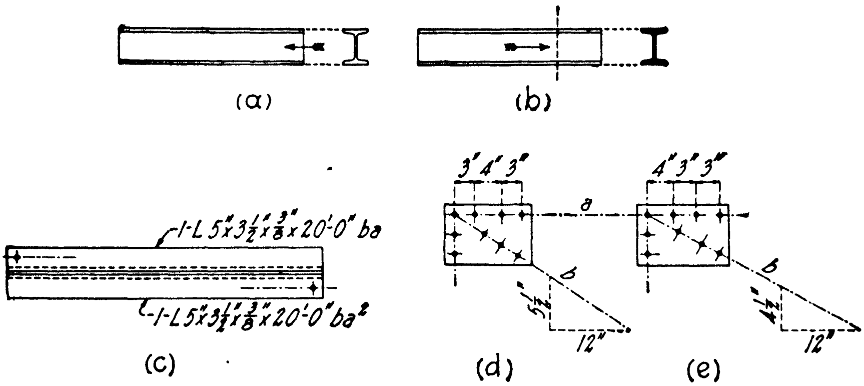


FIG. 19.

Connection Plates.—In detailing connection plates wherever bevel for holes on lines "b," (d) and (e) Fig. 19, is different, spacing for holes on lines "a" should be made different to prevent plates from being interchanged.

Writing Angles.—In writing angles give the longer leg first, 1-L 6″ × 4″ × ½″ × 10′-0½″.

Writing Plates.—In writing plates the width of the plate is given in inches, the thickness in inches, and the length in ft. and in.; 2-Pl. 48″ × ⅜″ × 15′-0¾″. A length of 9 in. should be written 0′-9″ and not 9″. The width of a plate is the dimension at right angles to the length of the member, while the length of a plate is the dimension parallel to the length of the member to which the plate is attached; except that for lacing bars, tie plates and other universal mill plates 6 inches and less in width the least dimension is taken as the width of the member, and for splice plates the width is the dimension at right angles to the splice.

Writing Sections.—Sections are written as follows: 1-I 12″ @ 40 lb. × 16′-3½″.

Miscellaneous.—Bevels may be shown as so many inches in 12″, (a) Fig. 20; or where convenient the total lengths may be given as in (b) Fig. 20. The latter method is the better as it assists the checker and the templet maker.

The maximum amount that one leg of an angle can be bent is 45°. For a greater bend than 45° a bent plate shall be used, (c) Fig. 20.

The center to center length of stiff laterals should be not less than $\frac{1}{16}$ in. short.

Do not use 2 sizes of rivets in the same leg, or same angle, or same piece unless absolutely necessary.

Where unequal legged angles are used mark the width of one leg of the angle on the leg.

Where heavy laterals are spliced in the middle by a plate, ship the plate riveted to one angle only.

Do not countersink rivets in long pieces unless absolutely necessary.

Do not draw any more of a member than necessary, and do not dimension the same piece several times.

Revising Drawings.—When drawings have been changed after having been first approved, they must be marked, Revised (give date of revision).

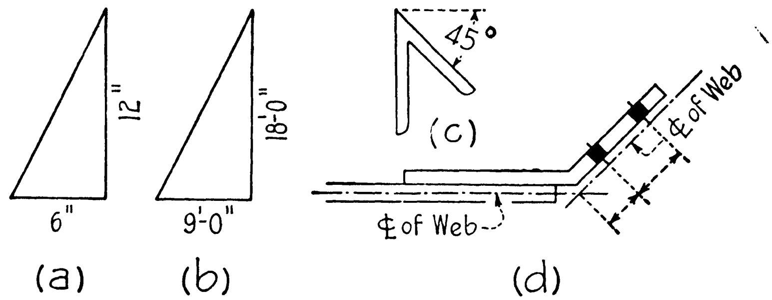


FIG. 20.

Measuring Angles.—All measurements on angles are to be made from the back of the angle, and not from the edge of the flange. The center to center distance between open holes should always be given for each piece that is shipped separate, in order that the inspector can check the piece.

Width of Angles.—The widths of the legs of angles are greater than the nominal widths, unless the angle has been rolled with a finishing roll. The over-run for each leg is equal to the nominal width of the leg plus the increase in thickness of leg made by spreading the rolls. For example finishing rolls are used for rolling $3'' \times 3''$ angles with a thickness of $\frac{1}{4}''$. The actual length of the leg of a $3'' \times 3''$ angle is as follows: angle $3'' \times 3'' \times \frac{1}{4}''$, leg $3''$; angle $3'' \times 3'' \times \frac{5}{16}''$, leg $3\frac{1}{16}''$; angle $3'' \times 3'' \times \frac{3}{8}''$, leg $3\frac{1}{8}''$; angle $3'' \times 3'' \times \frac{1}{2}''$, leg $3\frac{1}{4}''$; angle $3'' \times 3'' \times \frac{5}{8}''$, leg $3\frac{3}{8}''$.

The over-run of Pencoyd angles are given in Table 27, Part II; and the over-run of Pennsylvania Steel Company's angles are given in Table 28, Part II.

POINTS TO BE OBSERVED IN ORDER TO FACILITATE ERECTION.—The first consideration for ease and safety in erection should be to so arrange all details, joints and connections that the structure may be connected and made self-sustaining and safe in the shortest time possible. Entering connections of any character should be avoided when possible, notably on top chords, floorbeam and stringer connections, splices in girders, etc. When practicable, joints should be so arranged as to avoid having to put members together by entering them on end, as it is often impossible to get the necessary clearance in which to do this. In all through spans floor connections should be so arranged that the floor system can be put in place after the trusses or girders have been erected in their final position, and vice versa, so that the trusses or girders can be erected after the floor system has been set in place. All lateral bracing, hitch-plates, rivets in laterals, etc., should, as far as possible, be kept clear of the bottoms of the ties, it being expensive to cut out ties to clear such obstructions. Lateral plates should be shipped loose, or bolted on so that they do not project outside of the member, whenever there is danger of their being broken off in unloading and handling. Loose fillers should be avoided, but they should be tacked on with rivets, countersunk when necessary.

In elevated railroad work, viaducts and similar structures, where longitudinal girders frame into cross girders, shelf angles should be provided on the latter. In these structures the expansion

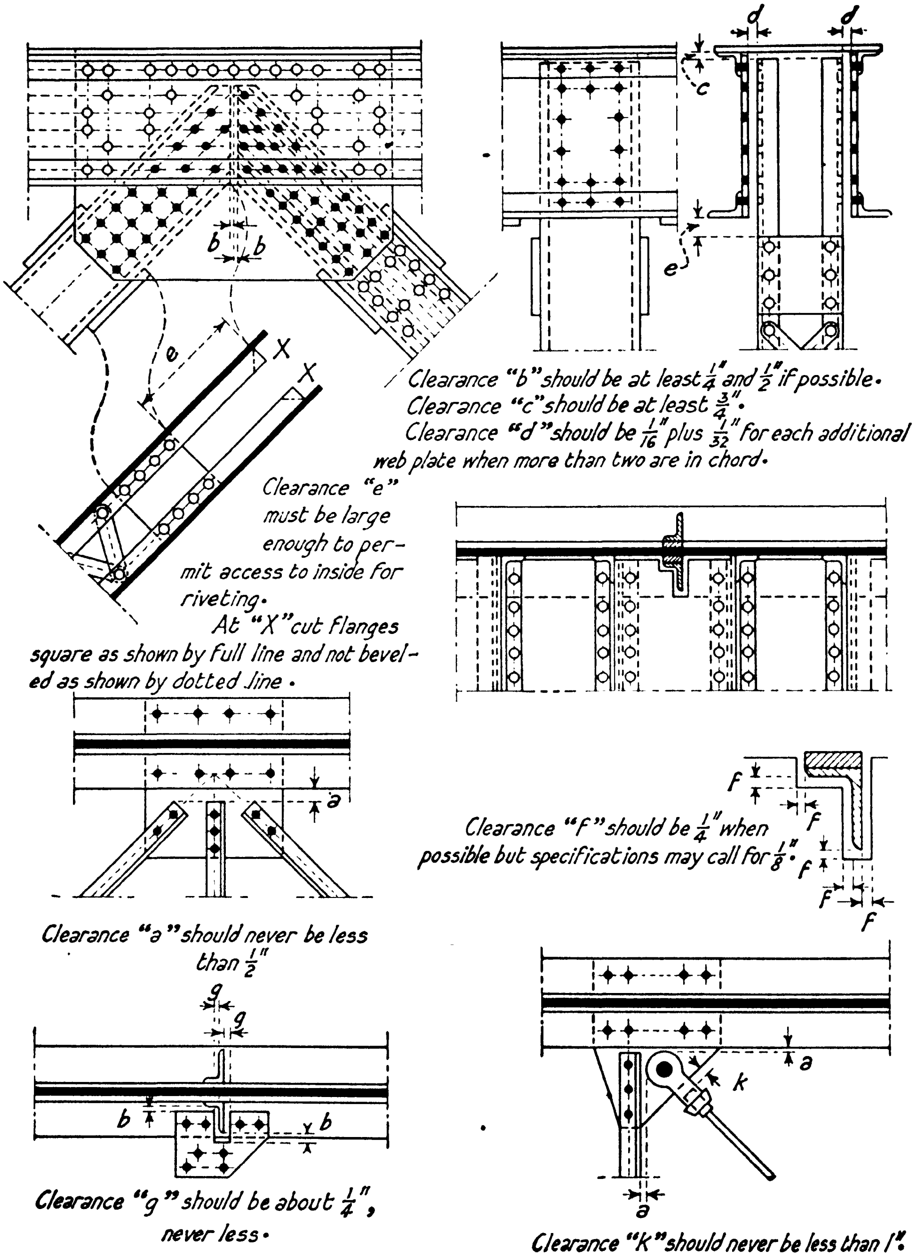
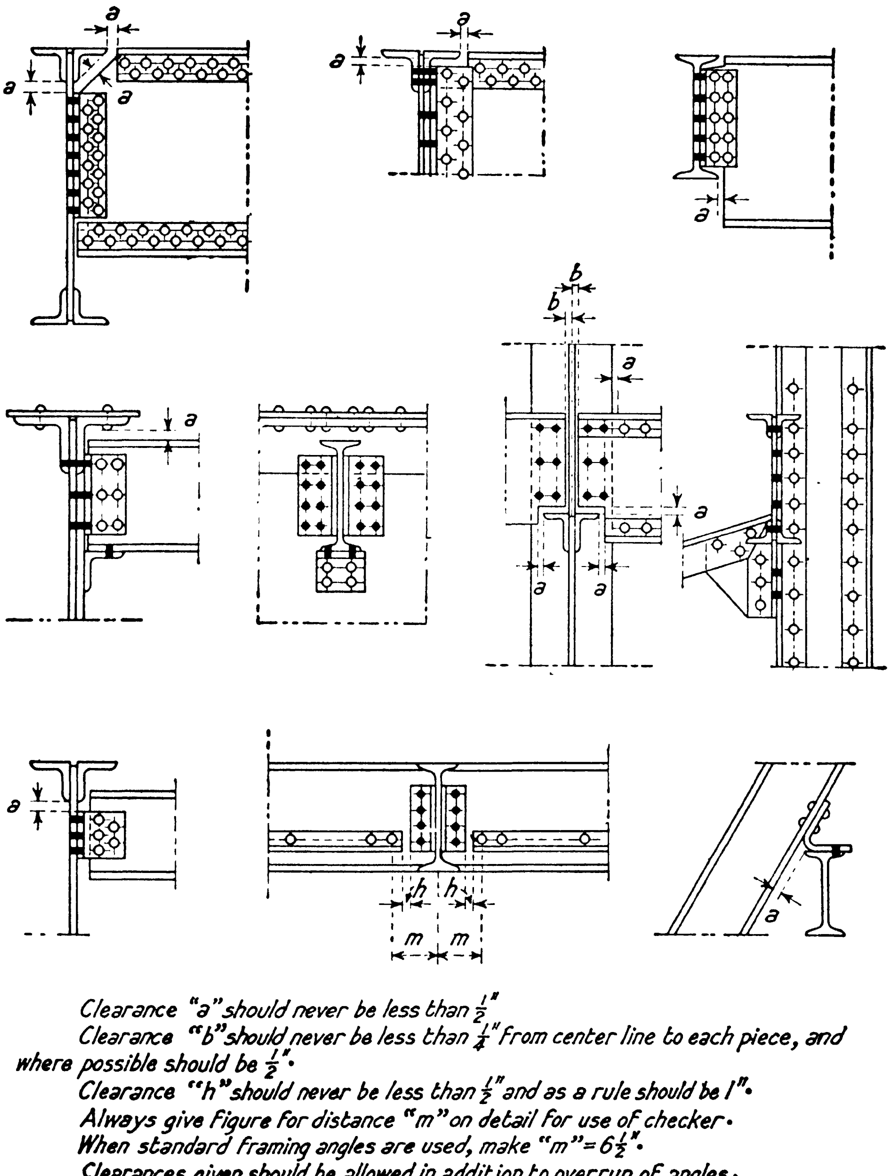
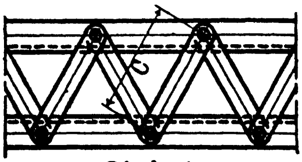


FIG. 21. CLEARANCE STANDARDS. AMERICAN BRIDGE COMPANY.

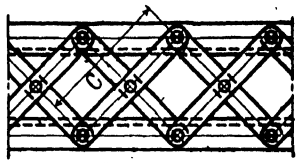


Clearance "a" should never be less than $\frac{1}{2}''$
Clearance "b" should never be less than $\frac{1}{4}''$ from center line to each piece, and where possible should be $\frac{1}{2}''$.
Clearance "h" should never be less than $\frac{1}{2}''$ and as a rule should be $1''$.
Always give figure for distance "m" on detail for use of checker.
When standard framing angles are used, make "m" = $6\frac{1}{2}''$.
Clearances given should be allowed in addition to overrun of angles.

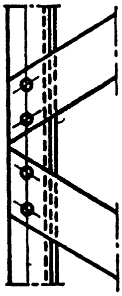
FIG. 22. CLEARANCE STANDARDS. AMERICAN BRIDGE COMPANY.



Style 1.

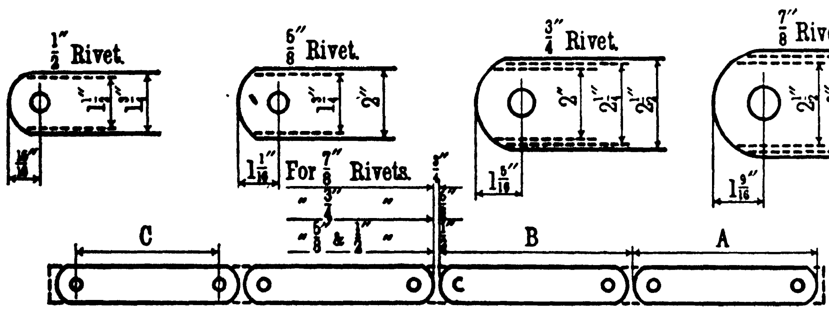


Style 2.



Style 3.

Maximum Distance C in feet and inches for given thickness T of bar				
T Thickness of Bar	Single Lacing		Double Lacing	
	C=40 T	C=50 T	C=60 T	C=75 T
⅝"	2'- 1"	2'-7¼"	3'- 1½"	3'-10⅞"
9/16	1-10½	2-4⅛	2- 9¾	3- 6 3/16
½	1- 8	2-1	2- 6	3- 1½
7/16	1- 5½	1-9⅞	2- 2¼	2- 8 13/16
⅜	1- 3	1-6¾	1-10½	2- 4⅛
5/16	1- 0½	1-3⅝	1- 6¾	1-11 7/16
¼	10"	1-0½	1- 3	1- 6¾



Length to be added to Distance C								
Width of Bar in Inches	For finished length A				For ordered length B			
	½"	⅝"	¾"	⅞"	½"	⅝"	¾"	⅞"
3				3⅛"				3⅛"
2¾				3⅛				3⅛
2½			2⅝"	3⅛			3⅛"	3⅛
2¼			2⅝				3⅛	
2		2⅛"	2⅝			2⅛"	3⅛	
1¾	1⅝"	2⅛			2⅛"	2⅛		
1½	1⅝				2⅛			

FIG. 23. STANDARDS FOR LACING BARS. AMERICAN BRIDGE COMPANY.

joints should be so arranged that the rivets connecting the fixed span to the cross girder can be driven after the expansion span is in place. In viaducts, etc., two spans, abutting on a bent, should be so arranged that either span can be set in place entirely independent of the other. The same thing applies to girder spans of different depth resting on the same bent. Holes for anchor-bolts should be so arranged that the holes in the masonry can be drilled and the bolts put in place after the structure has been erected complete.

In structures consisting of more than one span a separate bed-plate should be provided for each shoe. This is particularly important where an old structure is to be replaced; if two shoes were put on one bed plate or two spans connected on the same pin, it would necessitate removing two old spans in order to erect one new one. In pin-connected spans the section of top chords nearest the center should be made with at least two pin-holes. In skew spans the chord splices should be so located that two opposite panels can be erected without moving the traveler. Tie plates should be kept far enough away from the joints and enough rivets should be countersunk inside the chord to allow eye-bars and other members being easily set in place. Posts with channels or angles turned out and notched at the ends should be avoided whenever possible.

ORDERING MATERIAL.—Bridge Work.—Ordinarily plates less than 48 in. wide are ordered U. M. (universal mill or edge plates), but when there is no need for milled edges and prompt delivery is essential specify either U. M. or sheared. Never order widths in eighths. Flats and universal (edge) plates over 4 in. in width should be ordered in even inches, flats under 4 in. should be ordered by ½ in. variation in width. Flats ¼ in. and under in thickness are very difficult to secure from the mills and should be avoided if possible.

Rolling mills are allowed a variation of ¼ in. in width of plates, over or under, and a variation of ⅜ in. in length, over or under, from the ordered width or length. Rolling mills are allowed a variation of ⅜ in. over or under the ordered length of beams, channels, angles, zees, etc. An extra price is charged for cutting to exact length. See Chapter XIII.

Allow $\frac{1}{16}$ in. in thickness for planing plates 2 ft. 6 in. square or less, ⅛ in. for plates more than 2 ft. 6 in. square, and ⅛ in. for columns; chords and girders which have milled ends are ordered ¼ in. longer than the finished dimensions.

Web plates should be ordered ½ in. less than the back to back of flange angles unless a less clearance is specified. Web plates should preferably be ordered in even inches and the distance back to back of angles made in fractions.

When angles, beams or channels are bent in a circle allow 9 in. to 12 in. for bending.

Bent plates should be ordered to the length of the outside of the bend.

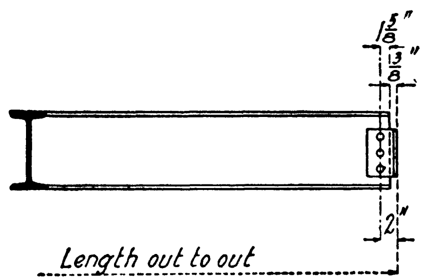


FIG. 24. BEAMS BETWEEN COLUMNS.

Large gusset plates, large plates with angle cuts, etc., should be ordered as sketch plates, when the amount of waste if ordered rectangular will exceed 20 per cent. Mills will not make re-entrant cuts in plates or shapes.

In ordering lacing bars add $\frac{3}{16}$ in. to the finished length and order in multiple lengths.

ORDERING MATERIAL.—Building Work.—Order beams in foundation neat length.

Order beams framing into beams ⅜ in. short for each end, see Fig. 24.

Order main column material $\frac{3}{4}$ in. long for milling both ends (this takes care of permissible variation in length of plus or minus $\frac{3}{8}$ in. as well as the milling).

Order girder flange angles and plates 1 in. long.

Order girder web plates $\frac{1}{2}$ in. short, where end connections are used.

Order girder web plates neat length, where end connections are not used.

Order girder web plates $\frac{1}{2}$ in. less in width than back of flange angles.

Order stiffener angles $\frac{1}{4}$ in. long.

Order fillers under stiffeners neat length.

Add $\frac{3}{16}$ in. to each lacing bar and order in multiple lengths.

SHAPES AND PLATES MOST EASILY OBTAINED.—The ease with which different commercial sizes of shapes and plates may be obtained from the rolling mill varies with the mill and with the demand. Where any section is in demand rollings are frequent and the orders are promptly filled, while the order for a section not in demand may have to wait a long time until sufficient orders have accumulated to warrant a special rolling.

The following list of plates and sections is fairly accurate, the list varying from time to time.

Plates.—Plates most easily obtained.

Width, In.	Thickness, In.	Width, In.	Thickness, In.
$1\frac{1}{2}$	$\frac{3}{16}$ and $\frac{1}{4}$	5	$\frac{1}{4}$ and up
$1\frac{3}{4}$	$\frac{3}{16}$ and $\frac{1}{4}$	6	$\frac{1}{4}$ and up
2	$\frac{3}{16}$ and $\frac{1}{4}$	7	$\frac{1}{4}$ and up
$2\frac{1}{4}$	$\frac{1}{4}$ and up	8	$\frac{1}{4}$ and up
$2\frac{1}{2}$	$\frac{1}{4}$ and up	9	$\frac{1}{4}$ and up
3	$\frac{1}{4}$ and up	10	$\frac{1}{4}$ and up
$3\frac{1}{2}$	$\frac{1}{4}$ and up	12	$\frac{1}{4}$ and up
4	$\frac{1}{4}$ and up	14	$\frac{1}{4}$ and up

Over 14 in. in width it is immaterial what width of plate is specified.

Squares and Rounds.—Squares and rounds most easily obtained.

Rounds, $\frac{5}{8}''$, $\frac{3}{4}''$, $\frac{7}{8}''$, $1''$, $1\frac{1}{4}''$, $1\frac{1}{2}''$.

Squares, $\frac{3}{4}''$, $\frac{7}{8}''$, $1''$, $1\frac{1}{4}''$, $1\frac{1}{2}''$.

All other sizes are liable to cause delay.

Beams.—Sizes of I-Beams which can be obtained most readily.

Depth.	Weight.		
6″	12.5 lb.		
8″	17.5 lb.	$20\frac{1}{2}$ lb.	
10″	25.4 lb.	30 lb.	
12″	31.8 lb.	35 lb.	40 lb.
15″	42.9 lb.	50 lb.	60 lb.
18″	54.7 lb.	60 lb.	70 lb.
20″	65.4 lb.	80 lb.	
24″	79.9 lb.	90 lb.	100 lb.

Sizes of I-Beams which may be used but for which prompt deliveries may not be expected.

Depth.	Weight.	
5″	10 lb.	
7″	15.3 lb.	
9″	21.8 lb.	25 lb.

Beams of weights different from the above can always be obtained from the mills but not so readily as those given. Beams of minimum section can always be obtained more readily than heavier sections.

Channels.—Channels which can be most readily obtained from the mills.

Depth.	Weight.
6″	8.2 lb.
8″	11.5 lb. 18¾ lb.
10″	15.3 lb. 20 lb. 25 lb.
12″	20.7 lb. 25 lb. 30 lb.
15″	33.9 lb. 40 lb. 50 lb.

Sizes which may be used but for which prompt deliveries cannot be expected.

Depth.	Weight.
5″	6.7 lb.
7″	9.8 lb.
9″	13.4 lb.

Channels of weights different than those given above can always be obtained at the mills but not so readily as those given. Channels of minimum section can always be obtained more readily than heavier sections.

Angles.—Angles most easily obtained from the mill.

Even legs.—2½″ × 2½″; 3″ × 3″; 3½″ × 3½″; 4″ × 4″; 6″ × 6″.

Uneven legs.—2½″ × 2″; 3″ × 2½″; 3½″ × 3″; 4″ × 3″; 5″ × 3½″; 6″ × 4″.

Angles which may be used but for which prompt deliveries cannot be expected.

Even legs.—2″ × 2″; 2¼″ × 2¼″; 5″ × 5″; 8″ × 8″.

Uneven legs.—3″ × 2″; 3½″ × 2½″; 4″ × 3½″; 6″ × 3½″.

Angles 4″ × 3½″; 5″ × 4″; 7″ × 3½″ and 8″ × 6″ are very difficult to obtain.

To obtain prompt deliveries as few sizes and shapes as practicable should be used for any contract. For example if 6″ × 4″ angles are used 6″ × 3½″ should be avoided, and vice versa.

Tees.—If possible the use of Tees should be confined to 3″ × 3″ × ⅜″ and 2″ × 2″ × 5/16″, and even these sizes are uncertain of delivery.

Zees.—The delivery of zees is uncertain and will depend upon special rollings, which do not occur frequently. The following sizes are the most used, and are therefore most easily obtained.

Web.	Thickness.
3″	¼″, 5/16″ and ⅜″
4″	¼″, 5/16″ and ⅜″
5″	5/16″, ⅜″ and ½″.
6″	⅜″, ½″, ⅝″, ¾″, ⅞″ and 1″

Stock Material.—The Pennsylvania Steel Company carries the following material in stock in 30 ft. lengths for use in its structural plant.

Angles, Even Legs.

6″ × 6″ × 7/16″ and ½″
4″ × 4″ × ⅜″ and 7/16″
3½″ × 3½″ × ⅜″ and 7/16″
3″ × 3″ × 5/16″, ⅜″ and 7/16″

Angles, Uneven Legs.

6″ × 4″ × ⅜″, 7/16″ and ½″
5″ × 3½″ × ⅜″, 7/16″ and ½″
4″ × 3½″ × 5/16″ and ⅜″
3½″ × 3″ × 5/16″ and ⅜″
3″ × 2½″ × 5/16″ and ⅜″

Plates.

20″ × ⅜″ and ½″
18″ × ⅜″ and ½″
16″ × ⅜″ and ½″
15″ × ⅜″ and ½″
14″ × ⅜″ and ½″
13″ × ⅜″ and ½″
12″ × ⅜″, 7/16″ and ½″
10″ × ⅜″ and 7/16″
9″ × ⅜″

Flats.

7″ × ⅜″
6″ × ⅜″ and ½″
3½″ × ⅜″, ½″ and ⅝″
3″ × ⅜″ and 7/16″
2½″ × ⅜″ and 7/16″
2¼″ × 5/16″ and ⅜″
2″ × ¼″ and 5/16″

Lengths and Widths of Plates.—The maximum sizes and lengths of shapes and plates as rolled by the Carnegie Steel Company and the Illinois Steel Company are given in Table I to Table VII, inclusive.

TABLE I.

MAXIMUM LENGTHS OF SHAPES; CARNEGIE STEEL CO.

I Beams:—	
$24''$ to $12''$	75 ft.
$10''$ to $5''$	70 "
$4''$ and $3''$	50 "
Channels:—	
$15''$ to $12''$	75 ft.
$10''$ standard	70 "
$10''$ special	80 "
$9''$ to $5''$	70 "
$4''$ and $3''$	50 "
Tees:—	
$5''$ to $1''$	50 ft.
Zees:—	
$6''$ and $5''$	70 ft.
$4'' \times \frac{3}{4}''$	65 "
$4'' \times \frac{11}{16}$ and under	70 "
$3''$	70 "
Deck Beams:—	
$10''$	45 ft.
$9''$ to $7''$	65 "
$6''$	60 "
Bulb Angles:—	
$10''$ to $7''$	65 ft.
$6''$	60 "
$5''$	65 "
Angles (Even Legs):—	
$8'' \times 8''$	120 ft.
$6'' \times 6'' \times 1''$ to $\frac{7}{8}''$	80 "
$6'' \times 6'' \times \frac{13}{16}''$ and under	90 "
$5'' \times 5''$	85 "
$4'' \times 4''$	90 "
$3\frac{1}{2}'' \times 3\frac{1}{2}''$	90 "
$3'' \times 3''$	75 "
$2\frac{3}{4}'' \times 2\frac{3}{4}''$	50 "
$2\frac{1}{2}'' \times 2\frac{1}{2}''$	50 "
$2\frac{1}{4}'' \times 2\frac{1}{4}''$	50 "
$2'' \times 2''$	50 "
$1\frac{3}{4}'' \times 1\frac{3}{4}''$ to $\frac{5}{8}'' \times \frac{5}{8}''$	50 "

Angles (Eneven Legs):—	
$8'' \times 6''$	80 ft.
$7'' \times 3\frac{1}{2}'' \times 1''$ to $\frac{7}{8}''$	80 "
$7'' \times 3\frac{1}{2}'' \times \frac{13}{16}''$ to $\frac{7}{16}''$	85 "
$6'' \times 4'' \times 1''$ to $\frac{3}{4}''$	85 "
$6'' \times 4'' \times \frac{11}{16}''$ and under	90 "
$6'' \times 3\frac{1}{2}'' \times 1''$ to $\frac{7}{8}''$	80 "
$6'' \times 3\frac{1}{2}'' \times \frac{13}{16}''$	85 "
$6'' \times 3\frac{1}{2}'' \times \frac{3}{4}''$ and under	90 "
$5'' \times 4''$	90 "
$5'' \times 3\frac{1}{2}'' \times \frac{7}{8}''$	75 "
$5'' \times 3\frac{1}{2}'' \times \frac{13}{16}''$	80 "
$5'' \times 3\frac{1}{2}'' \times \frac{3}{4}''$ and under	90 "
$5'' \times 3''$	90 "
$4\frac{1}{2}'' \times 3'' \times \frac{13}{16}''$	50 "
$4\frac{1}{2}'' \times 3'' \times \frac{3}{4}''$	55 "
$4\frac{1}{2}'' \times 3'' \times \frac{11}{16}''$	60 "
$4\frac{1}{2}'' \times 3'' \times \frac{5}{8}''$	65 "
$4\frac{1}{2}'' \times 3'' \times \frac{9}{16}''$	70 "
$4\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$ and under	80 "
$4'' \times 3\frac{1}{2}''$	90 "
$4'' \times 3'' \times \frac{13}{16}''$	85 "
$4'' \times 3'' \times \frac{3}{4}''$ and under	90 "
$3\frac{1}{2}'' \times 3'' \times \frac{13}{16}''$	60 "
$3\frac{1}{2}'' \times 3'' \times \frac{3}{4}''$	65 "
$3\frac{1}{2}'' \times 3'' \times \frac{11}{16}''$	70 "
$3\frac{1}{2}'' \times 3'' \times \frac{5}{8}''$	75 "
$3\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$ and under	80 "
$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{11}{16}''$	55 "
$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{8}''$	60 "
$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{9}{16}''$	65 "
$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{2}''$	70 "
$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{7}{16}''$	80 "
$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$ and under	90 "
$3\frac{1}{4}'' \times 2''$	50 "
$3'' \times 2\frac{1}{2}''$ to $1\frac{3}{8}'' \times 1''$	50 "

TABLE II.

MAXIMUM LENGTHS OF MATERIAL; ILLINOIS STEEL CO. (SOUTH WORKS).

Angles:—	
All angles	100 ft.
I Beams:—	
All I Beams up to 15 in.	75 ft.
15 I Beams 42.9 lb. to 55 lb.	75 "
15 I Beams 60 lb. to 75 lb.	62 "
15 I Beams 80 lb.	60 "
15 I Beams 90 lb.	50 "
15 I Beams 100 lb.	45 "
Channels:—	
All Channels	75 ft.

In case it is absolutely essential to have any of the above material in lengths longer than

shown, it will be necessary to take the matter up with the mill to ascertain whether same can be obtained.

TABLE III.

Rectangular and Circular Plates—Carbon Steel.

Sheared Plates, One-fourth Inch and Over, Extreme Sizes.

Carnegie Steel Company.

Thickness, In.	Weight, Lb. per Sq. Ft.	Widths and Lengths in Inches										Diam., In.
		128	126	120	114	108	102	96	90	84	78	
$\frac{1}{4}$	10.20				175	250	280	300	330	375	400	115
$\frac{5}{16}$	12.75			240	270	320	360	380	420	440	460	120
$\frac{3}{8}$	15.30	220	240	270	320	365	380	410	450	500	550	130
$\frac{7}{16}$	17.85	240	270	300	360	370	410	430	460	510	550	130
$\frac{1}{2}$	20.40	260	270	320	365	400	450	480	510	550	580	130
$\frac{9}{16}$	22.95	260	270	330	373	420	470	500	530	570	600	130
$\frac{5}{8}$	25.50	260	300	350	390	450	500	520	540	600	620	130
$\frac{11}{16}$	28.05	260	300	360	420	450	500	520	540	600	620	130
$\frac{3}{4}$	30.60	260	300	360	400	450	490	520	540	600	620	130
$\frac{13}{16}$	33.15	260	300	340	385	440	490	510	530	600	620	130
$\frac{7}{8}$	35.70	260	300	330	375	440	480	510	530	600	620	130
1	40.80	250	300	300	340	440	460	500	530	580	600	130
$1\frac{1}{8}$	45.90	250	300	300	330	410	440	450	500	550	580	130
$1\frac{1}{4}$	51.00	240	270	300	310	380	400	420	490	530	550	130
$1\frac{1}{2}$	61.20	220	230	260	280	330	320	340	420	440	480	130
$1\frac{3}{4}$	71.40	200	200	220	240	280	270	300	380	380	410	130
2	81.60	180	180	190	210	240	240	260	320	330	360	130
$2\frac{1}{4}$	91.80	150	160	170	190	210	210	230	280	295	320	130

Thickness, In.	Weight, Lb. per Sq. Ft.	Widths and Lengths in Inches										Diam., In.
		72	66	60	54	50	48	42	36	30	24	
$\frac{1}{4}$	10.20	430	475	525	530	530	530	530	530	530	530	115
$\frac{5}{16}$	12.75	480	500	560	550	575	575	550	550	550	580	120
$\frac{3}{8}$	15.30	600	600	620	620	620	620	600	580	600	600	130
$\frac{7}{16}$	17.85	600	630	630	640	640	640	600	580	600	600	130
$\frac{1}{2}$	20.40	610	630	630	640	640	640	600	580	630	600	130
$\frac{9}{16}$	22.95	620	640	640	640	640	640	600	580	630	600	130
$\frac{5}{8}$	25.50	620	640	640	640	640	640	600	580	600	600	130
$\frac{11}{16}$	28.05	620	640	640	640	640	640	600	580	600	580	130
$\frac{3}{4}$	30.60	620	640	640	640	640	640	600	580	600	580	130
$\frac{13}{16}$	33.15	620	640	640	640	640	640	600	580	570	550	130
$\frac{7}{8}$	35.70	620	640	640	640	640	640	600	580	550	550	130
1	40.80	600	630	630	640	640	640	580	580	520	530	130
$1\frac{1}{8}$	45.90	580	620	620	640	640	640	580	580	520	500	130
$1\frac{1}{4}$	51.00	550	600	600	600	600	600	560	560	520	450	130
$1\frac{1}{2}$	61.20	530	600	600	600	600	600	540	540	470	430	130
$1\frac{3}{4}$	71.40	450	490	550	550	550	550	540	540	430	380	130
2	81.60	400	440	480	500	500	500	500	500	400	350	130
$2\frac{1}{4}$	91.80	350	390	420	450	450	450	450	450	300	200	130

Plates 48″ wide and under can also be rolled on Universal Mills.
For greater length and Universal Mill Sizes, see Universal Mill Plate Table.
Plates of greater dimensions than shown above may be submitted for special consideration.

TABLE IV.

RECTANGULAR AND CIRCULAR PLATES—CARBON STEEL.

Sheared Plates, Three-sixteenth Inch, Extreme Sizes.
Carnegie Steel Company.

Thickness, In.	Weight, Lb. per Sq. Ft.	Widths and Lengths in Inches										Diam., In.
		90	84	78	72	70	68	66	64	60	54–24	
$\frac{3}{16}$	7.65	270	320	345	375	390	400	420	450	470	480	90

Plates of greater dimensions than shown in above table may be submitted for special consideration.

TABLE V.

RECTANGULAR UNIVERSAL PLATES—CARBON STEEL.

Universal Mill Plates, One-fourth Inch and Over, Extreme Sizes.
Carnegie Steel Company.

Thickness, In.	Weight, Lb. per Sq. Ft.	Widths and Lengths in Inches										
		48–46	45–41	40–36	35–31	30–26	25–20	19–17	16–15	14–12	11	10–6½
$\frac{1}{4}$	10.20						1020	1020	1020	1020	540	540
$\frac{5}{16}$	12.75	1020	1020	1140	1260	1320	1320	1080	1080	1080	600	600
$\frac{3}{8}$	15.30	1200	1200	1320	1380	1380	1380	1080	1080	1080	900	840
$\frac{7}{16}$	17.85	1320	1320	1380	1380	1380	1380	1080	1080	1080	900	840
$\frac{1}{2}$	20.40	1380	1380	1380	1380	1380	1380	1080	1080	1080	1020	840
$\frac{9}{16}$	22.95	1380	1380	1380	1380	1380	1380	1080	1080	1080	1020	840
$\frac{5}{8}$	25.50	1380	1380	1380	1380	1380	1380	1080	1080	1080	1020	840
$\frac{3}{4}$	30.60	1353	1357	1363	1372	1380	1380	1080	1080	1080	900	840
$\frac{7}{8}$	35.70	1160	1163	1169	1177	1188	1203	1080	1080	1080	900	840
1	40.80	1015	1018	1023	1030	1039	1052	1080	1080	1080	900	840
$1\frac{1}{8}$	45.90	903	905	910	916	924	936	1080	1080	1080	840	840
$1\frac{1}{4}$	51.00	812	814	818	824	832	842	1071	1080	1080	840	840
$1\frac{3}{8}$	56.10	738	740	744	749	756	766	973	1080	1080	840	840
$1\frac{1}{2}$	61.20	677	679	682	687	693	702	892	1059	1080	840	840
$1\frac{5}{8}$	66.30	625	626	629	634	640	648	823	978	1080	840	840
$1\frac{3}{4}$	71.40	580	581	584	588	594	601	765	908	1038	720	720
$1\frac{7}{8}$	76.50	541	543	545	549	554	561	714	847	968	660	720
2	81.60	507	509	511	515	519	526	669	794	907	600	720

Plates of greater dimensions than shown in above table may be submitted for special consideration.

TABLE VI.

Rectangular Plates—Nickel Steel.

Sheared Plates, One-fourth Inch and Over, Extreme Sizes.

Carnegie Steel Company.

Thickness, In.	Widths and Lengths in Inches														
	102	96	90	84	78	72	66	60	54	50	48	42	36	30	24
$\frac{1}{4}$						240	240	260	280	280	280	280	280	260	260
$\frac{5}{16}$					260	260	270	300	310	310	340	340	340	310	310
$\frac{3}{8}$		280	340	390	420	450	500	500	500	500	480	450	450	430	430
$\frac{7}{16}$	260	300	360	400	430	480	520	520	520	520	500	490	490	480	480
$\frac{1}{2}$	270	320	380	420	460	485	520	520	520	520	500	490	490	480	480
$\frac{9}{16}$	270	320	380	420	460	485	520	520	520	520	500	490	490	480	480
$\frac{5}{8}$	270	300	355	390	440	480	520	520	520	520	500	500	500	480	450
$\frac{11}{16}$	260	300	355	390	440	460	490	500	500	500	500	500	480	480	450
$\frac{3}{4}$	260	300	355	390	440	450	460	500	500	500	500	500	480	480	450
$\frac{13}{16}$	260	300	355	390	440	440	460	480	500	500	500	500	480	460	440
$\frac{7}{8}$	260	300	355	390	440	440	460	480	480	480	480	480	480	450	440
1	260	290	320	370	400	430	440	460	480	480	480	480	440	420	420
$1\frac{1}{8}$	250	270	295	330	375	400	410	420	440	440	440	440	440	420	420
$1\frac{1}{4}$	240	260	290	315	330	350	360	380	390	400	400	420	420	400	400
$1\frac{1}{2}$	230	260	290	290	310	330	350	370	390	390	390	390	380	380	360
$1\frac{3}{4}$	220	230	250	270	300	310	330	350	370	390	390	360	340	340	320
2	210	230	250	260	290	295	310	330	350	370	370	340	320	320	290

All sizes of Rectangular Nickel Steel Plates given in above table under ½″ thick should be specified to gage only. Plates ½″ thick and over can be rolled to either gage or weight per square foot.

Design Drawings for Steel Structures.

Drawings.—Designs shall be made on standard sized sheets. A scale of ¼ in. to 1 ft. shall be a minimum, a larger scale being used if practicable. Give such distances on both plan and cross-section that the dimensions of either can be understood without reference to the other.

Designs of Mill Buildings.

Loads.—All roof loads, snow loads, wind loads, floor loads, wheel loads and spacing for cranes, and in case of bins, the weight per cubic foot and the angle of repose of the material shall appear on the design drawings.

Diagrams.—Draw as many sections as are necessary to show all transverse bents and trusses, a plan of lower chord bracing, and views to indicate framing and side views when necessary to give location of doors and windows. When a sectional view is shown, always mark the location of the sections on the plan. When two buildings frame into each other the design should always indicate the framing for the connections, drawing additional sections if required.

Stresses.—The stresses in all members of transverse bents, trusses and latticed and plate girders, and the loads on all main building columns shall be given on the design drawings. Give maximum bending moment and maximum shear in all crane girders, plate girders, and floor girders and columns. Maximum shear and bending moment shall be given for all stringers or I-Beams used as floor or crane girders.

Notes.—Material (whether O. H. (open-hearth) or Bessemer, soft, medium or structural steel); specifications (name and date; size of rivets and holes, reamed or punched full size).

Angle Members.—In all cases where two unequal legged angles are used as main members, show the direction in which the outstanding legs are turned by giving the dimension of the leg appearing in elevation, or by exaggerating the longer leg.

TABLE VII.

RECTANGULAR PLATES—NICKEL STEEL.

Universal Mill Plates, One-fourth Inch and Over, Extreme Sizes.

Carnegie Steel Company.

Thickness, In.	Widths and Lengths in Inches										
	48–46	45–41	40–36	35–31	30–26	25–20	19–17	16–15	14–12	11	10–6¼
$\frac{1}{4}$							660	660	660	540	540
$\frac{5}{16}$	540	540	600	660	720	780	780	780	780	600	600
$\frac{3}{8}$	720	720	780	840	960	960	1020	1020	1020	900	840
$\frac{7}{16}$	840	840	960	1020	1080	1080	1020	1020	1020	900	840
$\frac{1}{2}$	960	960	1080	1140	1200	1200	1020	1020	1020	1020	840
$\frac{9}{16}$	960	960	1080	1140	1200	1200	1020	1020	1020	1020	840
$\frac{5}{8}$	900	900	1020	1080	1140	1140	1000	1000	1020	1020	840
$\frac{3}{4}$	840	840	960	1020	1080	1080	1000	1000	1020	900	840
$\frac{7}{8}$	780	780	840	960	960	960	1000	1000	1000	900	840
1	720	750	780	816	840	900	1000	1000	1000	900	840
$1\frac{1}{8}$	640	667	693	725	744	800	1000	1000	1000	840	840
$1\frac{1}{4}$	575	600	624	652	672	720	1000	1000	1000	840	840
$1\frac{3}{8}$	525	545	567	593	600	655	970	1000	1000	840	840
$1\frac{1}{2}$	480	500	520	544	540	600	890	1000	980	840	840
$1\frac{5}{8}$	444	461	480	502	504	554	820	978	980	840	840
$1\frac{3}{4}$	410	428	445	466	480	514	765	908	980	720	720
$1\frac{7}{8}$	384	400	416	435	444	480	710	847	968	660	720
2	360	375	390	408	420	450	670	794	908	600	720

All sizes of Rectangular Nickel Steel Plates given in above table under ½″ thick should be specified to gage only. Plates ½″ thick and over can be rolled to either gage or weight per square foot.

Sections.—Give sections of all members used in the structure. Whenever two or more columns or other members in different locations have the same section, either note it, or mark the section on each one. For a column of special make-up show a cross section.

Dimensions.—The following dimensions should be given: (1) Height of lower chord of trusses from floor level; (2) elevation of top of crane rail with clearance; (3) distance c. to c. of crane rail with clearance; (4) distance b. to b. of angles of all main columns; (5) pitch of trusses or height of same at heel and slope of upper chord; (6) width and height of ventilator; (7) length of bays; (8) distance c. to c. of building columns; (9) location and size of stacks; (10) location and size of openings and circular ventilators; (11) thickness of all walls, and relation to center line of columns.

Windows.—Give size and number of lights and height of windows. Show location of all windows. State whether pivoted, sliding, counter-balanced or fixed, and whether continuous. State kind of glass.

Doors.—Give dimensions (width by height) and state whether wood or steel, swinging, lifting, rolling or sliding. State style of track, hangers and latch.

Louvres.—Note depth on design, and whether wood or metal, fixed or pivoted. If metal give gage and kind of same.

Corrugated Steel.—Give gage and kind of all corrugated sheeting, painted or galvanized; method of fastening, lining, etc.

Gutters and Conductors.—Show gutters, conductors and downspouts where necessary and give size and kind and thickness of metal, methods of fastening, etc.

Circular Ventilators.—Show location on design and note size and kind.

Roofing.—Give kind of roofing material, and thickness of sheathing when used.

Notes.—Note on design the section of: (a) Purlins and form where trussed; (b) girts; (c) sag rods; (d) lateral bracing; (e) end columns; (f) window posts; (g) door posts.

Connections.—In making a design be sure that all clearances and connections with adjoining structures are properly provided for and that all dimensions necessary for detailing of same are given on the design.

Designs of Plate Girder Bridges.

Loads.—Give assumed dead, live and wind loads, and show diagram of wheel loads.

Diagram and Views.—Show an elevation of girder with stiffeners, a plan with lateral bracing, and a half end view and a half intermediate section.

Stresses.—Give maximum bending moments and maximum shears, maximum stresses, required and actual net area of flanges, noting number of rivets deducted, and required net and actual gross areas of webs.

Dimensions.—The following dimensions should appear on all plate girder designs. Distance b. to b. of end angles, or distance out to out of girders, c. to c. of bearings, back wall to back wall, or c. to c. of piers, b. to b. of flange angles, spacing of girders and track stringers, base of rail to masonry, end of steel to face of back wall, angle of skew if any, and grade of base of rail.

For girder bridges on curves give the curvature and super-elevation of outer rail and distance from top of masonry to base of low rail. Give elevation of grade and of masonry on a vertical line through center of end bearing.

Rivet Spacing.—Note on the elevation of girders the spacing of rivets connecting flange angles to web, changing spacing at stiffener points. Give number of rivets in single shear for end connections of all laterals and cross frames.

Shoes and Pedestals.—Give maximum reaction, required and actual area of masonry plate, with allowable pressure on masonry. Note size of bed plate, and show in position with location of holes for anchor bolts. Note size and number of rollers for expansion pedestal, and also whether pedestal is built, cast iron or steel.

Expansion Points.—Mark fixed and expansion points and show whether pedestals or bearing plates are to be used.

Stiffeners.—Show end and intermediate stiffeners on elevation of girder, giving sections and stating whether fillers are used, or stiffeners crimped.

Super-elevation.—If the bridge be on a curve, show how the super-elevation of the outer rail is to be cared for, whether by tapering ties, or changing height of pedestal or masonry plate.

Track.—Show track in place, noting such information as size and notching of ties and guard timbers and manner of connecting timber deck to the girder. For through girder always show clearance diagram with dimensions.

Notes.—(a) Material (whether O. H. (open-hearth) or Bessemer, soft, medium or structural steel); (b) specifications (name and date); (c) size of rivets and holes, reamed or punched full size.

Designs of Truss Bridges.

Loads.—Always give the following assumed loads on the stress sheets.

Dead Loads.—(a) Weight of track in lb. per lin. ft. of track; (b) weight of trusses and bracing per lin. ft. of bridge; (c) weight of stringer and stringer bracing per lin. ft. of bridge; (d) weight of floorbeams per lin. ft. of bridge.

Live Load.—(Diagram of wheel loads.)

Wind Load.

Diagrams.—In general, the design shall show an elevation of the truss, plan of top lateral bracing, plan of bottom lateral bracing and stringer bracing, half end view showing portal, half intermediate view, or as many intermediate views as are necessary to show intermediate sway frames. The end view shall show track in place with information similar to that for plate girders. The design of a pin-connected bridge shall show the sizes of pins and the arrangement of the members at all panel points.

Stresses.—Give the stresses in all members of trusses as follows: D. L. (Dead Load); L. L. (Live Load); I. (Impact); C. (Curvature); W. (Wind Stresses). Also total stresses.

Always use the minus sign for tensile stress and the plus sign for compressive stress. Compute and give traction stresses for viaduct towers.

For stringers and floorbeams give the bending moment and shear and stresses in the same manner as for plate girders.

General Dimensions.—The most important dimensions are, number of panels and length, depth of truss at every panel point if upper chord is curved, distance c. to c. of trusses, distance base of rail to masonry, distance center of end pin to masonry, distance c. to c. of end pins and face to face of masonry, or c. to c. of piers. If the bridge be on a curve, give the degree and show direction of curvature, the distance of base of low rail to masonry, and the super-elevation of outer rail. Note that greater clearances are required on curves. Show the clearance line and line of base of rail in the elevation of truss.

Compression Members.—Give the actual unit stress, the allowable unit stress, radius of gyration, moment of inertia, actual and required area, eccentricity and cross-section.

Tension Members.—Give allowable and actual stresses, the required and actual net area. For built sections give number of holes deducted for rivets in obtaining net area, and radius of gyration.

Sections.—Give section of every member and thickness of all gusset plates. Always give size of lacing bars, and state whether single or double lacing is required.

Built Sections.—On all built sections give depth of section, and in using plate and angle sections, make the web ½ in. less in width than the depth of section.

Angles with Unequal Legs.—In any member composed of one or more angles with unequal legs, show clearly the direction in which the long or short leg is turned.

Rivets.—Note the number of rivets to be used for end connections of all members, and give the number of rivets in single shear required at end connection of track stringers.

Shoes or Pedestals.—Give maximum reaction, required and actual area of masonry plate, with allowable pressure on masonry. Note size of bed plate, and show in position with location of holes for anchor bolts. Note size and number of rollers for expansion pedestal, and also whether pedestal is built, cast iron or steel.

Camber.—The amount of camber should be shown on the design.

Notes.—Same as for Plate Girders.

Points to be Observed in Detailing to Improve and Simplify the Design.
American Bridge Company.

1. **Types of Details.**—Other things being equal, use details requiring the smallest number of pieces. The simplest detail is usually the cheapest and best.

When beams connect to the flange face of plate and angle columns, it is preferable to use web connection angles riveted to the beams.

When beams connect to the web face, it is preferable to use seat angles. When columns are of light section and may be readily tilted or sprung, it is preferable to use web connection angles riveted to the beams for both web and flange faces of columns.

For beams connecting to girders, use a seat (without stiffeners) when possible and a side clip riveted on girders. Do not provide any holes for connecting beams to seats. When the seat requires stiffeners, it is better from a shop point of view to frame beams with regular connection angles, unless the stiffeners under the seat can be made to take the place of regular stiffeners required on the plate girders.

On columns composed of channels 10 in. and under, use details that require no stiffeners for connecting to web face, and as far as possible details that eliminate the use of all connections to the webs of any size of channels. On box columns eliminate as far as possible details that require rivets to be driven through the cover plates alone, as such rivets require an additional operation in riveting and assembling.

2. **Metal over Metal.**—In details which are designed to transfer concentrated loads by direct bearing, care must be used to get metal over metal. Intervening plates should not be counted upon to distribute the load unless an analysis of stress proves them equal to it.

3. **Slabs.**—In the transmission of pressure, steel slabs may be used when rolled plates cannot be obtained of sufficient thickness to insure an even distribution of stress. (Plates thicker than the published maximum up to the tabulated minimum thickness of slabs can usually be obtained by omitting the specification of physical test.) Rolled slabs are cheaper than forged slabs, built bases, or castings; but forged slabs and castings can sometimes be obtained more quickly, for this reason their use is permitted when the question of delivery is an important consideration. The limiting sizes of rolled and forged slabs are as given in Table VIII.

TABLE VIII.

Sizes of Steel Slabs.

	Carnegie (Rolled)	Pencoyd (Rolled)	Gary (Forged)
Minimum thickness	4″	3″	....
Maximum thickness	15″	15″	10″
Minimum width	15″	15″	....
Maximum width	50″	25″	36″
Minimum length	3′ 0″	3′ 0″	....
Maximum length	20′ 0″	20′ 0″	15′ 0″

Limiting sizes are also subject to the equipment of the fabricating plant. The limiting weight at Carnegie Mills is 7,000 lb., at Pencoyd Mills is 8,000 lb. and at Gary Forge is 18,000 lb. When slabs are used have as few thicknesses as possible.

4. **Deck Girder Laterals.**—Square deck plate girder spans should have an even number of panels in the lateral system, so that the girders can be made symmetrical about center. Skew deck plate girder spans should have an odd number of panels in the lateral system, so that the girders can be turned end for end.

5. **Lintels.**—When angle lintels are used, it is not necessary to rivet the angles together unless called for. Plain angles should be used. Preference should be given to plain angle over cast iron lintels. When I beams and channels are used as lintels, no anchors need be provided.

6. **Draw for Diagonals.**—For transverse, longitudinal, and lateral diagonal bracing of one or more angles, allow the following draw:

For lengths up to and including 10 ft., nothing.
" " 11 ft. to 20 ft., inc., shorten $\frac{1}{16}$ in.
" " 21 " " 35 " " " $\frac{1}{8}$ "
" " over 35 ft. " $\frac{3}{16}$ "

Drop sixteenths, but do not vary from above more than $\frac{1}{16}$ in. These deductions are to be made in the length of laterals for deck or through truss spans from the lengths computed and proper allowances made for camber. Whether laterals attach to stringers or not does not affect the rule.

7. **Mullions.**—Mullions and other members running from floor to floor on office buildings should be detailed with vertically slotted holes in one end to prevent their taking loads for which they are not designed. The slotted holes should be in the connection angles and not in the main material.

8. **Bases in Concrete.**—When built column bases and grillage girders are imbedded in concrete, if acceptable to architect, use full head rivets.

9. **Upset Rods.**—Rods $\frac{7}{8}$ in. or less in diameter should not be upset; when necessary to obtain sufficient section, increase the diameter. Use cold rolled threads when possible. Avoid the use of clevises on rods $1\frac{1}{8}$ in. in diameter and less. Rods over $1\frac{1}{8}$ in. in diameter should be connected by clevises.

10. **Water Pockets.**—Avoid forming water pockets in structures exposed to the weather, or provide a drain hole where it will effectually drain the pocket.

11. **Lateral Connections.**—Unless it increases the size of lateral plate disproportionately, omit the lugs from light lateral angles which require no more than five rivets.

12. **Castings.**—Before detailing castings, refer to records and find out if old patterns can be used (preferably without alteration). Use old patterns when possible. In similar castings, when there are variations in heights not exceeding 1 inch, take up the variation by thickening the top or bottom. Holes in castings which connect to steel work should be drilled, not cored. All cast shoes or bases must be planed on top and bottom except those which are to be grouted when the bottom need not be planed. Weep holes should be put in all castings where water is liable to collect and freeze.

13. **Anchor Bolt Types.**—For anchor bolts built in masonry, use a rod with nut at each end (not a forged head at one end), see Standards for Detailing. Use cold rolled threaded or swedged anchor bolts where masonry is to be drilled for bolts.

14. **Crane Rail Splices.**—Crane rails should not be spliced at the same points as the girders or beams supporting them.

15. **Purlin and Girt Spacing.**—When sheathing is used, purlin spacing should suit stock sizes of lumber. When corrugated sheeting is used, purlin and girt spacing should be arranged to suit standard lengths of corrugated steel.

16. **Templet Shop to Determine Lengths.**—In built member of 5 feet depth or less, with latticed web between chords, not parallel, the draftsman should neglect figuring lengths and inclinations, leaving them for the templet shop to determine.

17. **Mark Ends of Girders.**—Mark the ends of girders which go at the same end of bridge " X," or " North," or " to New York," so that every advantage of knowing this relation may be had in the shop and in loading for shipment. Add a note calling attention to this mark.

18. **Holes in Checkered Plates.**—Avoid countersinking holes in checkered plates. Use flat headed bolts, or screw headed bolts, in places where the nuts cannot be turned with a wrench.

19. **Buckle Plates.**—The field riveting of buckle plate floors is sometimes cheapened by making the drainage holes large enough (about $1\frac{1}{2}$ in. diameter) to allow the passage of rivets to the sticker, who is below the floor. If this method of erection meets with the approval of the Erection Department, endeavor to have the details approved with these large drainage holes.

20. **Three Processes of Galvanizing.**—The hot process consists in dipping the piece in molten spelter; the electric process consists in coating by electric contact; and sherardizing consists in placing the piece in an air-tight tube filled with zinc oxide and then heated to the required temperature, to be removed after cooling. The hot process is used in coating structural material and castings; the other two processes for bolts and small castings.

21. **Cleaning before Galvanizing.**—Before galvanizing, all mill scale, rust, grease and paint should be removed from the surface. This is done by immersing the piece first in a bath containing a solution of sulphuric acid to remove the objectional matter and then in water to wash off the acid. Finally the piece is dipped in a solution of ammonium chloride after which it is ready to be galvanized.

22. **Detailing Material to be Galvanized.**—The most desirable material for the hot process is straight plain angles and shapes, preferably not more than 24 ft. 9 in. long (the length of the pot), but by turning the piece it is possible to galvanize a 31 ft. length. Built up members should not exceed 24 ft. 9 in. in length. Bent work decreases the output and increases the cost of galvanizing. The pot is 40 in. deep and 30 in. wide. Pieces wider than the depth of the pot must

be turned both in the pickling and spelter tanks. Before proceeding with the fabrication of any material to be galvanized whose dimensions approach the maximum, the Shiffler Plant should be consulted. Work riveted in the shop is more expensive to galvanize than plain material Gusset plates should not be riveted to angles. Field connections generally should be bolted (not riveted) with galvanized bolts.

23. **Bolts for Galvanized Work.**—Unless otherwise specified bolts should be sherardized, as hot galvanizing fills the threads with spelter. If hot galvanizing is specified for bolts, the threads should be cut deeper than standard or re-cut after galvanizing.

24. **Marking Galvanized Work for Identification.**—The mill and shop marks should be put on with tailor's chalk instead of paint. Erection marks should be stamped with a steel stencil in a definite place on each piece.

25. **Weight of Galvanized Material.**—In figuring the weight of galvanized structural material, it should be assumed that galvanizing adds to the weight approximately .08 lb. per sq. ft. of surface covered.

26. **Car Load Shipments of Material.**—Detail work as far as possible to obtain full car load lots for shipment. On contracts for tanks, smoke stacks, tubular piers or pipes, which are shipped west of the Mississippi River, if there is any question as to whether they should be shipped knocked down or riveted up, the matter should be referred at once to the Division Engineer. There is a great difference in freight rates, particularly on less than car load lots, on shipments into that territory.

27. **Riveting Watertight and Oiltight Work.** The diameter, pitch and arrangement of rivets are to be determined by the thickness of the plates or bars which they connect. The maximum spacing for rivets in watertight work is 4 diameters. The maximum spacing for rivets in oiltight work is $3\frac{1}{2}$ diameters. The minimum spacing for rivets is $2\frac{1}{2}$ diameters. All seams should be chain riveted. The distance between lines of rivets in seams or butts, if chain riveted, should not be less than 3 diameters; if staggered, not less than $1\frac{1}{2}$ diameters. Avoid staggering rivets if possible. Longitudinal seams should be single riveted when plates are $\frac{3}{8}$ in. thick or less, double riveted when plates are $\frac{7}{16}$ in. to 1 in. thick, and triple riveted when plates are over 1 in. thick. Overlapped butts or single butt straps should be double riveted when plates are $\frac{3}{8}$ in thick or less, triple riveted when plates are $\frac{7}{16}$ in. to $\frac{9}{16}$ in. thick, and quadruple riveted when plates are $\frac{5}{8}$ in. or $\frac{3}{4}$ in. thick. When plates are over $\frac{3}{4}$ in. thick butts should have double straps triple riveted. Butt straps should be $\frac{1}{16}$ in. thicker than the plates they connect.

28. **Punching Plates.**—When specifications require that plates be punched from the faying surfaces (surfaces in contact in completed work), it is preferable, except for plates $\frac{5}{16}$ in. and less in thickness to sub-punch and ream all holes in order to avoid turning the plates at the punch. This clause of the specification is often waived on thin plates, as for these plates its observance accomplishes nothing.

29. **Edge Distances for Oiltight and Watertight Work.**—The distance from center of hole to extreme point of bevel sheared edge of plate should be not less than $1\frac{1}{2}$ diameters, and when calked not more than $1\frac{3}{4}$ diameters of the rivet.

30. **Calking.**—Calking edges of $\frac{3}{8}$ in. plates or less are to be bevel sheared. If allowed by customer, plates over $\frac{3}{8}$ in. thick are to be sheared straight. The bevel on plates $\frac{3}{16}$ in. to $\frac{3}{8}$ in. thick, inclusive, should be about 30 degrees, and on plates over $\frac{3}{8}$ in. thick should be about $\frac{3}{16}$ in. Ordinarily plates less than $\frac{3}{16}$ in. thick are not calked, but canvas or lampwick is placed in the seam to make it watertight. Ambridge plant can bevel-shear plates $\frac{5}{8}$ in. thick or less.

Points to be Observed in Detailing to Simplify Erection.
American Bridge Company.

1. **General.**—In designing details, care should be taken to arrange all joints and connections so that the work can be built at the shop with a minimum cost in labor and material, and can be erected most economically and with a minimum risk.

2. The sequence of erection should be considered in making the details.

3. In bridge work, connections should be detailed so that spans can be made self-sustaining and safe in the shortest possible time.

4. Top chord sections in each panel are put in place after the posts and bars for that panel are erected. In heavy work it is especially desirable that the details be arranged so that these chord sections can be lifted above the posts and set directly into place without being moved endways or sideways. For such work, plates connecting adjoining sections should be shipped loose.

5. It is usually customary, when local conditions permit, to put the floor system in place first and erect the trusses afterward. This method of procedure has a great many advantages over that of raising the trusses first, viz.: there is a great saving in false work, as longer panels can be used; it permits bents to be placed directly under the panel points and the new floor system to be used for carrying traffic and running out material for the trusses; it permits the

posts to be bolted to the floorbeams and released from the tackles on the travelers; it fixes the exact position of the shoes on the piers so that the erection can proceed from the center toward either fixed or roller end, as may be preferred; it gives more opportunity for jacking up the spans to secure proper camber; and it requires a minimum amount of blocking.

6. Over dangerous streams where there is a possibility of loss during erection, it may be desirable to erect the trusses first. This brings as little material as possible on the false work. A minimum amount of material is thus endangered. Sometimes there are local conditions which make it imperative to erect the trusses first.

7. Therefore in all through truss spans, the floor connections should be so arranged that the floor system can be put in place after trusses have been erected in their final position, and vice versa, so that trusses can be erected after the floor system has been set in place.

8. For through plate girder spans, the stiffeners should be arranged so that floor system can be put in place without spreading the main girders. Also on through trough floor spans, wherever possible, details should be arranged so that trough floor can be put in position without spreading main girders.

9. In all work, as far as practicable, details should be arranged so that members can be swung into position without shifting from their final position members to which they connect. If this is impossible, place note on erection plan calling erectors' attention to this special feature.

10. Stiffeners to which cross frames or floorbeams connect should not be crimped, but have fillers. The outstanding legs should preferably be not less than 5 in., and never less than 4 in. Open holes in stiffener angles should be gauged so that the cross frames can be swung into place without spreading the main girders.

11. **Pin Spans.**—The sections of top chords nearest the center should be made with at least two full pin holes. In skew spans the top chord splices should be located so that the two opposite panels can be erected without moving traveler. In curved top bridges, the top chords should be designed so that each panel of truss can be erected and self-sustained before moving the traveler to the next panel.

12. **Pilot Nut Interference.**—When portals or top bracing would apparently interfere with the use of long pilot nuts, it is not necessary or desirable to ship short pilot nuts. These pieces are seldom, if ever, erected before the pins are driven.

13. **Clearances.**—See that ample clearances are allowed. In allowing clearances to cover shop variations for cutting, shearing and coping, any clearance less than $\frac{1}{8}$ in. is equivalent to no clearance. For riveted web members entering between chords, allow a total clearance of $\frac{1}{8}$ in. or $\frac{1}{16}$ in. on a side. For plates to be inserted between angles, allow a total clearance of $\frac{1}{8}$ in. For beams and girders with top and bottom connection angles, whether riveted or shipped loose, allow $\frac{1}{4}$ in. clearance between top and bottom angles. When beams are framed directly to the web at the upper floor of heavy two-story columns, to facilitate erection, the rivets above the connection should be countersunk or left open for field driving.

14. **Packing of Eyebars and Pin Plates.**—In pin connected bridges with eyebars 8 in. and under, allow $\frac{1}{16}$ in. clearance for each eyebar and an additional total clearance of not less than $\frac{1}{8}$ in. between the two sides of the chord. For eyebars over 8 in. up to and including 12 in., double the above figures. For eyebars over 12 in., use three times the allowance for bars 8 in. and under. When more than two pin plates are used on a member, allow $\frac{1}{32}$ in. additional for each pin plate.

15. **Clearance at Ends of Beams and Girders.**—For crane or floor girders, milled (or made exact), and for floorbeams framing between columns, girders or beams, allow $\frac{1}{16}$ in. at each end. For plate girders, not milled (or made exact), allow $\frac{1}{8}$ in. at each end. For all structures in which girders or beams occur in continuous lines over 150 ft. long, Plant Engineer should decide what precautions are necessary to prevent an increase or decrease in total extreme dimensions of the building. For a certain percentage of the connections, girders and beams may be cut short and fillers provided. Other means of accomplishing the result may be found advisable.

16. **Cross Frame Clearance.**—Cross frames should be made of such a depth or so detailed as to permit them to be swung into place without interfering with the rivet heads in flanges of main girders. When the cross frames of the deck span connect to the top and bottom flange of the girder, allow $\frac{1}{16}$ in. clearance at top and $\frac{1}{16}$ in. at bottom.

17. **Clearance for Diagonals.**—In erecting diagonals in pin spans, it is customary to connect them at the bottom first and then to swing them into position around the lower pins as centers. A clear path should be provided for pieces erected in this way.

18. **Anchor Bolt Clearances.**—Holes are generally $\frac{1}{4}$ in. larger than diameter of bolt.

Column resting directly on grillage:

Punched Hole in Column = Diameter of Anchor Bolt + $\frac{1}{4}$ in.
Punched Hole in Grillage = Diameter of Anchor Bolt + $\frac{1}{16}$ in.

Column resting on cast base:

Punched Hole in Column = Diameter of Anchor Bolt + $\frac{1}{4}$ in.
Drilled Hole in Base = Diameter of Anchor Bolt + $\frac{1}{16}$ in.

Column resting on steel slab:

Punched Hole in Column = Diameter of Anchor Bolt + $\frac{3}{8}$ in.
Drilled Hole in Slab = Diameter of Anchor Bolt + $\frac{3}{8}$ in.
Punched Hole in Girder = Diameter of Anchor Bolt + $\frac{1}{16}$ in.

For large anchor bolts or to meet special conditions, it may be desirable to have the holes larger. Special consideration should be given to such cases. When anchor bolts are to be put in after structure is erected, arrange details to allow drilling of holes with material in position.

19. **Movable Bridge Clearance.**—On draw spans and bascule bridges, the clearance between the moving member and the nearest stationary member should be at least 2 in., preferably more. Fascia plates between fixed and draw spans on highway work should be provided with means of vertical adjustment.

20. **Stagger Turnbuckles.**—Adjustable rods or bars placed close together should have sleeve nuts or turnbuckles staggered.

21. **Clear Rivet Heads.**—An interference frequently occurs both in the shop and field between the outstanding flange of some piece and rivet heads of the piece into which it connects. This should be avoided.

22. **Cut Flanges to Clear.**—Girders which frame into webs of columns should, when necessary, have their flanges notched to clear rivet heads in outstanding legs of columns. This will permit erection without spreading columns.

23. **Clearance Above Rail.**—Note whether the clearance shown on stress sheets for railroad bridges is from top or from base of rail.

24. **Clear Old Work.**—In detailing work adjacent to old work or to walls, see that the rivets can be driven when work is in place. Spandrel beams adjoining old walls should be detailed to swing into place from inside of building.

25. **Entering Connections.**—Entering connections should be avoided.

26. **Shifting Members to Drive Rivets.**—Work should be detailed so that members can be placed in final position before riveting is commenced. Exceptions should be noted on erection drawings.

27. **Clearance for Driving.**—All field connections should be examined to see that they can be driven after the structure is erected. Draftsmen should inform themselves of the sizes and types of pneumatic hammers in use and necessary working space required.

28. **Expansion Pockets.**—Girders and stringers which rest in expansion pockets should set back sufficiently to allow the insertion of the field rivets for the end connection of the adjacent fixed member, as both members are in place before the rivets are driven.

29. **Lateral Plates Clear Ties.**—On deck girder spans and on stringers in through spans, lateral plates and rivet heads should be kept low enough to clear the ties.

30. **Slotted Holes for Anchors.**—In both ends of plate girders less than 50 ft. use slotted holes, or holes of extra large diameter, for anchor bolts.

31. **Length of Slots.**—The length of slots in expansion details should be sufficient to allow for a movement in either direction equal to the combined effect of temperature change, stress deformation and inaccurate workmanship.

32. **Erection Seats.**—It is not necessary to provide erection seats for beams framing into columns or girders, except when beams frame in opposite on web of plate and angle columns or girders and take the same open holes. Erection seats should usually be provided for plate girders framing into girders or columns. When erection seats are provided, a clearance of $\frac{1}{8}$ in. should be left between the bottom of the girder and the seat angle to allow for inaccuracies in setting the seat. No clearance should be provided when open holes are reamed to metal templet.

33. **Stitch Loose Fillers.**—Fillers should be shop riveted to members. Avoid loose fillers where possible.

34. **Stitch Loose Covers.**—When a long line of field rivets occurs in the cover, web or reinforcing plates of a column, chord or other built-up member, provide occasional rivets (countersunk if necessary) to keep the plate in contact with the main section of the member.

35. **Parts Reversible.**—If practicable arrange details of a member so that it may be reversed in erection.

36. **Parts not Reversible.**—When members are nearly but not quite symmetrical and it is possible to erect them reversed or inverted, mark the piece to indicate the way it should enter the structure. Thus: " Mark This End ' Toward Center ' " or " Mark ' This Side Up.' "

37. **Marking Directions.**—When the position in which a member is to be placed in a structure cannot readily be determined from the member itself and the erection plans, the sides or end of member should be marked showing direction in which member is to be set.

38. **Extra Field Work.**—All drilling and cutting to be done in the field should be clearly noted on the detail and erection drawings before final approval.

39. **Special Field Drilling.**—Sometimes it is advisable to drill certain holes in the field. This may occur in special cases where adjustment is needed or when the drawing room finds it

impracticable to locate a connection. In such cases the Plant Engineer should be consulted. A note should be added to the erection drawing calling attention to this special work.

40. **Holes for Tap Bolts.**—If tap bolts are to be used for field connections which transmit shear, the holes to be tapped should be drilled either in the shop or in the field, using the connecting piece as a templet. This avoids drifting, which destroys the threads. If the connections do not transmit shear, the drilling may be avoided by making the holes in the connecting piece large enough to provide for slight irregularity in spacing.

41. **Abutting Deck Spans.**—When two spans abut on a bent, as in a viaduct, details should be arranged so that either span can be set in place entirely independent of the other. The end cross frames should be detailed to be swung into place from the center.

42. **Holes for Auxiliary Work.**—Provide holes in steel work for connecting all auxiliary work, such as nailing strips, spiking pieces, skylight curbs, windows, doors, etc. The method of attaching auxiliary work to the steel work should be thoroughly understood.

43. **Replacing Old Bridges.**—In replacing an old bridge of more than one span, a separate bed plate should be provided for each shoe.

44. **Column Overrun.**—The overrun or packing out of cover plates on built up columns need be considered only when there are four or more cover plates on a face, in which case the distance out to out of covers should be figured $\frac{1}{8}$ in. more than the distance back to back of angles, plus the thickness of the covers.

45. **Anchor Bolts in Advance.**—Anchor bolts built into the masonry before the erection of the steel work on domestic and export work should be shipped in advance unless otherwise requested. Anchor bolts to be set after steel work is erected need not be shipped in advance.

SPECIFICATIONS FOR CAMBER OF TRUSSES AND GIRDERS.

AMERICAN BRIDGE COMPANY.

1. **Railroad Spans over 200 Ft. and Highway Spans over 250 Ft.**—For railroad spans over 200 ft., and highway spans over 250 ft., the distortion due to dead and live load should be computed, and length of all members modified so that lengths will be normal under these loads. The live load stress due to a uniform live load over the full length of span and not the maximum live load stress should be used. This uniform load should be such as would produce the same live load stress in center chords as given on stress sheet. Find the ratio between the sum of the dead and live load stress, and dead load stress in the center chords $= (DL + LL)/DL$. Multiply the dead load stress in each member by this ratio and find the distortion in length due to this stress. The normal lengths of the members should be figured, and for compression members should be increased, and for tension members decreased by an amount equal to the distortion. The lengths should also be corrected for play in pin holes. The camber of the truss resulting from the above changes in length and the pin play can be found graphically by what is known as a Williot Diagram. If a given amount of camber is to remain in the truss under dead and live load, the change in length of truss members from the normal, as outlined above, should be increased in the proportion that the camber under dead and live load stress has to deflection at center due to dead and live load. In figuring distortions, impact stresses should be neglected.

2. **Railroad Truss Spans up to 200 Ft.**—For railroad spans, up to and including 200 ft., trusses should be cambered by increasing the top chord length $\frac{1}{8}$ in. for each 10 ft. in length, unless specifications state that camber shall be made a proportional part of span. If the specifications give either the rate of top chord increase or the camber, the camber corresponding to the top chord increase corresponding to the camber, can be found from the equation $E = (8C \cdot H)/S \cdot N$ in which E = panel increase in in., C = camber in in., H = depth of truss in ft., S = length of span in ft., N = Number of panels. The above equation is derived from the equation of a circle, the posts being considered as radial.

3. **Highway Truss Spans up to 250 Ft.**—For highway spans up to and including 250 ft., the top chord lengths should be increased $\frac{3}{16}$ in. for each 10 ft. in length, unless specifications give some other amount or method of obtaining camber.

4. **Curved Top Chord Trusses.**—When trusses have varying depths, the panel increase as determined above is to be considered the panel increase at the center of the span and the increase in top chord panel length for any panel should have the same proportion to the panel increase at the center of the span, as the height of the truss at the center of the panel in question has to the height of the truss at the center of the span.

5. **Diagonals.**—The lengths of all truss diagonals and end-posts are to be computed, assuming each panel of the truss to be a true geometrical figure with the full panel camber increase added to the normal panel length of the top chord. For compression members of a pin connected truss, the length center to center of pin holes is to be the exact computed length, but for tension members $\frac{1}{32}$ in. should be deducted from the computed length as a correction for play in the pin holes. In the center panel of a riveted truss, both diagonals should have their computed cambered length reduced $\frac{1}{16}$ in. for draw.

6. **Deck Bridges.**—For deck bridges with vertical end posts, the length of end panel of floor and top chords should be reduced by one-half the top chord increase between the center of truss and panel point U_1. In figuring length of stringers for deck spans, add one-half of panel increase to normal panel length, and consider this the distance center to center of floorbeams. In figuring top laterals for deck or through bridges, the full panel increase should be used, and usual draw deducted from calculated length of diagonal.

7. **Camber Diagram.**—A camber diagram giving the ordinate at each panel point of the bottom chord from the horizontal to the "no load" line should be placed on the erection plan of all truss spans for export and for all spans of 250 ft. or more.

8. **Plate Girder Spans.**—No camber is to be used in plate girder spans unless required by customer. If camber is required, it is to be taken care of in such a way as to satisfy the customer and as best suits the plant where the work is fabricated.

9. **Roof Trusses.**—Roof trusses having spans of 75 ft. or under should not be cambered. The deflection of trusses should be considered when a definite clearance must be maintained for crane or other purposes, also when wall or other framing is attached to lower chord. Trusses over 75 ft. span may be cambered when considered necessary by Plant Engineer, the amount of camber being approximately equal to the deflection of truss under full load. For parallel chord or flat pitch trusses, camber produced by lengthening top chord $\frac{1}{8}$ in. in 10 ft. will approximately equal deflection under full load. Fink or other steep pitch trusses, if cambered at all, should be cambered by method of theoretical deformation. When roof trusses are cambered, the girts, bracing, etc., are figured on the assumption that the truss is fully loaded.

SHOP RIVET PERCENTAGES.

AMERICAN BRIDGE COMPANY.

1. For estimating weight of shop rivets, use the following percentages of the ordered weight of material; not including the weight of steel joists in highway bridges nor the weight of corrugated sheeting in mill buildings.

2. **Highway Bridges.**—Through pin-connected spans, with built floorbeams,

100 ft. and under	3 per cent.
Through pin connected spans, with built floorbeams, over 100 ft.	3.5 per cent.
Through pin-connected spans, with rolled floorbeams, 100 ft. and under	2.7 per cent.
Through pin-connected spans, with rolled floorbeams, over 100 ft.	3 per cent.
Deck riveted spans, with T-chords, 100 ft. and under	4.5 per cent.
Riveted pony truss spans, with box chords, 80 ft. and under	6.5 per cent.
Riveted pony truss spans, with T-chords, rolled beams 80 ft. and over	3 per cent.
Deck plate girder spans	5 per cent.
Through plate girder spans	6 per cent.
3. **Railroad Bridges.**—Single track deck plate girder spans	6 per cent.
Single track through plate girder spans	5.5 per cent.
Double track through plate girder spans	5 per cent.
Single track through riveted spans	6 per cent.
Double track through riveted spans	4 per cent.
Single track through pin-connected spans	4.5 per cent.
Double track through pin-connected spans	3.5 per cent.
Single track deck riveted spans	7 per cent.
Double track deck riveted spans	6 per cent.
Single traek deck pin-connected spans	5 per cent.
Double track deck pin-connected spans	4.8 per cent.
Single track viaducts with stiff bracing	5.5 per cent.
Double track viaducts with stiff bracing	4.8 per cent.
Trough floor	7 to 10 per cent.
4. **Steel Mill Buildings.**—Light roof construction and light mill buildings without crane runways	3 per cent.
Light mill buildings, with crane runways	4 per cent.
Heavy mill buildings, with crane runways	4.5 per cent.
Extra heavy mill buildings, with crane runways	5 per cent.
5. **Office Buildings.**—Riveted work	3 to 4 per cent.
Beams	$\frac{1}{2}$ to 1 per cent.

RAILWAY SHIPMENTS.

AMERICAN BRIDGE COMPANY.

1. **Shipping Height.**—Nearly all railroads can handle pieces at least 10 ft. 6 in. high (except pieces too long for a single car, when thickness of bolster, 12 in. to 15 in, must be included) and 6 ft. wide, or 10 ft. 3 in. wide and 7 ft. 3 in. high. In detailing pieces approaching these dimensions, Traffic Department should be consulted. Information required from Traffic Department should be obtained through Plant Manager's office.

2. **Shipping Lengths.**—Ordinary gondola and flat cars are from 34 ft. to 40 ft. long. Care should be taken to detail light pieces for single car shipment not more than 40 ft. long.

3. **Shipping Weights.**—Pieces exceeding 22 ft. in length, or too wide to be loaded through the side door of a standard 36 ft. box car, should be detailed so that they can be loaded on gondolas or flats. They can then be shipped in less than car load lots (first class rate). The minimum weights for which freight charges will be figured at either car load or less than car load rates are indicated in Table IX.

TABLE IX.

	East Miss. River Eastern Classification, lb.	West Miss. River Western Classification, lb.	Pacific Coast States Classification, lb.	South Mason & Dixon Line. East Miss. River Southern Classification, lb.
Less car load lots	1,000	5,000	5,000	4,000
Min. car load, bridge iron	36,000	36,000	40,000	30,000
Min. car load, girders and roof trusses	30,000	36,000	40,000	30,000
Min. load pieces requiring two cars	45,000	45,000	60,000	45,000
Min. load pieces requiring three cars	60,000	60,000	80,000	60,000
Min. load pieces requiring four cars	75,000	....	....	75,000

When more than four cars (three cars Western Classification) are used, the additional car or cars are considered as a new series; the series being determined by number of cars over which a continuous load extends.

CHAPTER XIII.

Estimates of Structural Steel.

GENERAL INSTRUCTIONS.—When an estimate of the structural steel in a structure is to be made the man in charge shall immediately examine all of the data furnished to see that he has sufficient information to make a satisfactory estimate. He shall fill out the data sheet completely, and then take off the quantities. Use only the standard estimate blanks for taking off material. The author has found the estimate blank below very satisfactory.

CROCKER & KETCHUM
Consulting Engineers
DENVER, COLO.

Estimate of Weight of 160-Ft. Span Highway Bridge   Sheet No 1 of 9

For Logan Irrigation Co.   Date Feb. 23, 1912

No. Pcs.	Description	Length		Weight Per Ft.	Weight: Main Members	Weight: Details	Weight: Complete Member	No. Req'd	Total Weight
	4 END POSTS	L_0	U_1	each	thus:-				
2	8" [s @ 11 1/4#	26	1 3/4	11.25	589				
1	Cov Pl. 12"×5/16"	26	5	12.75	337				
2	Bat. Pl. 12"×1/4"	1	0 1/2	10.20		21			
4	Hinge Pl. 6 1/2"×1/4"	0	8	5.33		16			
4	Pin Pl. 8"×3/8"	1	3	10.20		51			
2	Fill Pl. 6 1/2"×1/16"	0	6	1.38		2			
46	Lac. Brs 1 3/4×1/4"	1	1 1/2	1.49		83			
526	Riv. Hds. For 3/4" φ, per 100-			16.12		85			
					926	258	1184	4	4736

Number each page consecutively, and when all the quantities are totaled prepare a summary on the last page. Each sheet shall have the sheet number and also the total number of sheets in the estimate, for example 9 of 20. This will prevent the loss of a page. After the estimate is completely taken off another man shall check it. When checked the estimate shall be extended by the checker, each sheet being immediately totaled up as extended. The extensions shall then be checked by the original estimator, who also prepares a summary. The summary is then checked by the checker and the estimate is complete.

The estimate should be practically a condensed bill of material of the work, and should be so clearly made that a reference to the estimate will show at a glance the weight of all the principal pieces. Main and secondary trusses, main columns, girders, crane girders, etc., for buildings; and trusses, girders, floorbeams, etc., for bridges should be taken off separately, thus—1 truss, 6 required—and shall not be mixed together even though the correct weight is obtained. In making an estimate the following order will be found convenient

1. **MILL BUILDINGS.—Trusses.**—Top chords, lower chords, web members, purlin lugs, gusset plates, connection plates, splice plates, eave strut connections, knee braces and knee brace connections.

Ventilator Trusses.—Rafters, posts, web members, gusset plates, connections to trusses and purlin lugs.

Columns.—Column angles, web plate, base plate and angles, crane seat and cap. Base includes anchor bolts.

Crane Girders.—Flange angles, web plate, cover plates, end stiffeners, intermediate stiffeners, fillers, knee braces and knee brace connections. Rails, splice bars, clips and crane stops.

Miscellaneous.—Eave struts, lattice girders, purlins, girts, ridge struts, lower chord struts column struts, rafter bracing, lower chord diagonals, reinforcing angles for purlins used as rafter struts, and sag rods.

Miscellaneous Materials Not Structural Steel.—Corrugated steel roofing and siding, louvres, flashing and ridge roll, gutters, conductors, downspouts, ventilators, stack collars. Windows, doors, skylights, operating device, lumber, roofing, brick and concrete.

2. **OFFICE BUILDINGS.**—Floorbeams, girders, including all their connections not riveted to other members. Floors should be estimated separately using a multiplier if two or more are exactly alike.

Columns.—Columns including splices and connections riveted to the columns. If columns are of Bethlehem "H" sections, it should be so noted on the estimate summary. Estimate columns in tiers.

Miscellaneous, such as suspended ceilings, galleries, penthouses, lintels, curb-angles, canopies, etc.

3. **TRUSS BRIDGES.**—Truss members should be taken off separately in order that the estimate will show at a glance the weight of any main member. Never write off material for the trusses thus, "½—Truss—4 Req'd."

Stringers; floorbeams; portals; sway trusses; upper laterals; lower laterals: shoes, masonry plates, anchor bolts, etc.

A convenient order can easily be arranged for other structures.

INSTRUCTIONS FOR TAKING OFF MATERIAL.—Quantity estimates shall give the shipping weights, not shipping weights plus scrap. Pin plates, gusset plates, etc., shall be taken off as equivalent rectangular plates. Large irregular plates or small irregular plates which occur in larger numbers shall have the exact sizes shown in the estimate and should have their weights accurately calculated. All quantity estimates shall be made out with black drawing ink.

The following colored pencils shall be used in estimating:

Black.—In taking off quantities, all check marks on drawings or blue prints shall be made with a black pencil.

Red.—In checking "quantities taken off" all check marks on drawings, blue prints and data sheets shall be made with a red pencil.

Blue.—Blue pencils shall be used for checking extensions, also for making notes, corrections, alterations or additions on white prints or tracings.

Yellow.—All alterations, corrections or additions, on blue prints at the time of estimating shall be made with a yellow pencil.

All notes on blue prints or drawings in regard to alterations, corrections or additions shall be dated and signed by the person in charge of the estimate. In general all work shall be taken off in feet and inches. Lengths of bolts shall be given in feet and inches.

CLASSIFICATION OF MATERIAL.—In making the summary steel and iron should be classified as follows:

Bars, including plates 6 in. wide and under, rounds up to 3 in. in diameter and squares up to 3 in. on a side.

Plates (*a*) Flats over 6 in. wide up to and including 100 in., and ¼ in. thick and over.
(*b*) Flats over 100 in. wide up to and including 110 in.
(*c*) Flats over 110 in. wide up to and including 115 in.
(*d*) Flats over 115 in. wide up to and including 120 in.
(*e*) Flats over 120 in.
(*f*) Plates $\frac{3}{16}$ in. thick.
(*g*) Plates ⅛ in. thick.

(*h*) Plates checkered.
(*i*) Plates buckle.
Angles (*a*) Having both legs 6 in. wide or under.
(*b*) Having either leg more than 6 in. in width.
(*c*) Having both legs less than 3 in. in width.
Channels and I-Beams
(*a*) Channels and beams up to and including 15 in. in depth.
(*b*) Over 15 in. in depth.

If Bethlehem sections are used distinguish between "Bethlehem Special I-Beams" and "Girder Beams," and also regarding depths as above.

Zees.
Tees.
Rails (Separate rails under 50 lb. per yd., rails over 100 lb. per yd., and girder rails).
Rail Splices.
Iron Castings.
Steel Castings.
Nuts.
Clevises and *Turnbuckles.*
Pins, rounds from 3 in. diameter to 6¾ in. in diameter.
Forgings, rounds over 6¾ in. in diameter.
Bronze, *Lead*, etc.
Rivets and Bolts.

Rivet Heads.—Where the estimate is made from shop drawings the actual number of rivet heads shall be determined. The weight of rivet heads in per cent of the total weight of the other material is about as follows: Purlins, girts and beams, 2 per cent; trusses and bracing, 4 per cent; plate girders and columns of 4 angles and 1 pl., 5 per cent; plate girders and columns with cover plates, 6 per cent; box girders or channel columns with lacing, 7 per cent; trough floors, 8 to 10 per cent.

The rivet heads in highway bridges may be taken at 5 and 4 per cent of the total weight of steel exclusive of fence and joists for riveted and pin-connected trusses, respectively.

Bolts are usually taken off in the estimate when they occur, and entered as rivets. When bolts are under 6 in. in length, include bolts under the item "Bolts and Rivets." When over 6 in. in length, put the bolts under "Bars."

Miscellaneous Materials.—*Corrugated Steel.*—Always give the number of gage, whether painted or galvanized, and whether iron or steel. This remark also applies to louvres, flashing, ridge roll, gutters and conductors. State whether corrugated steel is for roofing or siding. Roofing shall be estimated in squares of 100 sq. ft., adding three feet on each end of building to the distance c. to c. of end trusses to allow for cornice. Allow one foot overhang at eaves. Siding shall be estimated in squares of 100 sq. ft., adding one foot at each end of building to allow for corner laps.

Louvres shall be estimated in sq. ft. of superficial area, stating whether fixed or pivoted.

Flashing shall be estimated in lineal feet and shall be taken off over all windows where corrugated sheathing is used on the sides of building, and under all louvres and windows in ventilators.

Ridge roll shall be estimated in lineal feet, adding one foot to the distance center to center of end trusses. Ridge roll is usually taken off the same gage as the corrugated steel roofing.

Gutters and conductors shall be estimated in lineal feet, the conductors usually being spaced from 40 to 50 ft., depending upon the area drained.

Circular ventilators shall be estimated by number, giving diameter and kind, if specified.

Stack collars shall be estimated by number, giving diameter of stack.

Windows shall be estimated in sq. ft. of superficial area, taking for the width the distance between girts. State whether windows are fixed, sliding, pivoted, counter-balanced or counter-weighted. State kind and thickness of glass and give list of hardware, and any thing else of a special nature.

Doors shall be estimated in sq. ft.; state whether sliding, lifting, rolling or swinging. Steel doors covered with corrugated steel shall be estimated by including the steel frame under steel and the covering with corrugated steel siding. State style of track, hangers and latch.

Skylights shall be estimated in sq. ft., giving kind of glass and frames.

Operating devices for pivoted windows or louvres shall be estimated in lineal feet.

Lumber shall be estimated in feet, board measure, noting kind. Note that lumber under 1 in. in thickness is classified as 1 in. Above 1 in. it varies by $\frac{1}{4}$ in. in thickness, and if surfaced will be $\frac{3}{8}$ in. less in thickness, i. e., $1\frac{3}{4}$ in. sheathing is actually $1\frac{3}{8}$ in. thick, but shall be estimated as $1\frac{3}{4}$ in. Lumber comes in lengths of even feet; if a piece 10 ft.–8 in. or 11 ft.–0 in. is required, a stick 12 ft.–0 in. long shall be estimated. In using lumber there is usually considerable waste depending upon the purpose for which it is intended. In estimating tongue and grooved sheathing 10 to 20 per cent shall be added for tongues and grooves and from 5 to 10 per cent for waste, depending upon the width of boards and how the sheathing is laid.

Composition roofing or slate shall be estimated in squares of 100 sq. ft., allowing the proper amount for overhang at eaves and gables and for flashing up under a ventilator or on the inside of a parapet wall.

Tile roofing or slate shall be estimated in squares of 100 sq. ft., adding 5 per cent for waste. Include in an estimate for tile roof, gutters, coping, ridge roll, plates over ventilator windows and plates under ventilator windows, these being estimated in lineal feet. Flat plates for the ends of ventilators shall be estimated in sq. ft.

Brick shall be estimated by number. For ordinary brick such as is used in mill building construction, estimate 7 brick per sq. ft. for each brick in thickness of wall, i. e., a 9 in. wall is two bricks thick and contains 14 brick for each sq. ft. of superficial area.

Always note whether walls are pilastered or corbeled and estimate the additional amount of brick required. If walls are plain, no percentage need be added for waste, but if openings such as arched windows occur add from 5 to 10 per cent.

Concrete shall be estimated in cubic yards. Walls or ceiling of plaster on expanded metal shall be estimated in squares of 100 sq. ft., noting thickness and kind of reinforcement. Reinforced concrete floors shall be estimated in sq. ft. of floor area, noting thickness and kind of reinforcement. Paving of all kinds is estimated in square yards, but the concrete filling under the pavement itself is estimated in cubic yards. Concrete floor on cinder filling is usually estimated in square yards, specifying its proportions.

ESTIMATE OF COST.—The different types of framed steel structures vary so much with local conditions and requirements that it is only possible to give data that may be used as a guide to the experienced estimator. The cost of steel frame structures may be divided into (1) cost of material, (2) cost of fabrication, (3) cost of erection, and (4) cost of transportation.

The costs of materials and labor have been abnormal for several years making cost data of relatively little value. Conditions are slowly returning to normal, but it is not possible at present to predict what the final base level will be. In the following discussion the costs are for prewar, 1914, *conditions unless the actual date is given.*

1. **Cost of Material.**—The price of structural steel is quoted in cents per pound delivered f. o. b. cars at the point at which the quotation is made. Current prices may be obtained from the Engineering News-Record, Iron Age or other technical papers. The present prices (1924) f. o. b. Pittsburgh, Pa., are about as follows:

TABLE I.

PRICES OF STRUCTURAL STEEL (1924) F. O. B. PITTSBURGH, PA., IN CENTS PER POUND.

Material.	Price in Cts. per Lb.
I-beams, 18 in. and over	2.60
I-beams and channels, 3 in. to 15 in.	2.50
H-beams, over 8 in.	2.75
Angles, 3 in. to 6 in. inclusive	2.50
Angles, over 6 in.	2.60

Zees, 3 in. and over	2.50
Angles, channels, and zees, under 3 in	2.50
Deck beams and bulb angles	2.80
Plates, structural, base	2.75
Corrugated steel No. 22, painted	4.00
Corrugated steel No. 22, galvanized	4.70
Steel sheets Nos. 10 and 11, black	3.10
Steel sheets Nos. 10 and 11, galvanized	4.00
Steel sheets No. 22, black	3.75
Steel sheets No. 22, galvanized	4.55
Bar iron, base	2.40
Rivets	2.75

COST OF FABRICATION OF STRUCTURAL STEEL*.—The cost of fabrication of structural steel may be divided into (*a*) cost of drafting, (*b*) cost of mill details, and (*c*) cost of shop labor.

(*a*) COST OF DRAFTING.—The cost of drafting varies with the character of the structure and with the shop methods of the bridge company. There are two general methods in common use for detailing steel structures, sketch details, and complete details (see Chapter XII). The cost of drafting varies with the method of detailing and the number of pieces to be made from one detail, and costs per ton may mean but little and be very misleading. The cost per standard sheet (24 in. × 36 in.) is more nearly a constant and varies from $15 to $25 per sheet. The following approximate costs, based on a total average charge of 40 cents per hour may be of value.

Mill and Mine Buildings.—Details of ordinary steel mill buildings cost from $2 to $4 per ton; details for headworks for mines cost from $4 to $6 per ton; details for churches and court houses having hips and valleys, cost from $6 to $8 per ton; details for circular steel bins cost from $1.50 to $3 per ton; details for rectangular steel bins cost from $2 to $4 per ton; details for conical or hopper bottom bins cost from $4 to $6 per ton.

Bridges.—Details of steel bridges will cost from $1 to $2 per ton where sketch details are used and from $2 to $4 per ton where the members are detailed separately.

Actual Cost of Drafting.—The details of the Basin and Bay State Smelter, containing 270 tons, cost $2 per ton.

The costs of making shop details for steel structures as given in the Technograph No. 21, 1907, by Mr. Ralph H. Gage, are given in Table II.

TABLE II.

COST OF SHOP DRAWINGS.

Character of Building.	Average Cost per Ton.
Entire skeleton construction, i. e., loads all carried to the foundation by means of steel columns	$1.45
Interior portion supported on steel columns; exterior walls carry floor loads and their own weight	1.22
Interior portion carried on cast iron columns; exterior walls support floor loads as well as their own weight	0.70
No columns and floorbeams resting on masonry walls throughout	0.85
Structure consisting mostly of roof trusses resting on columns	2.47
Structure consisting mostly of roof trusses resting on masonry walls	1.25
Mill buildings	2.56
Flat one-story shop or manufacturing buildings	0.74
Tipples, mining structures or other complicated structures	4.88
Malt or grain bins and hoppers	2.47
Remodeling and additions where measurements are necessary before details can be made	1.87

* Prewar, 1914, costs are given.

Mr. Gage makes the following comments on the cost of drafting: "The cost of drafting materials and blue prints was not included. There is always a noticeable decrease in cost of the details when the plans for the ironwork are made and designed by an engineer and separated from the general work. On the average it cost 35 per cent more to make shop drawings of the structural steel when the data were taken from the architect's plans than when the data were taken from carefully worked out engineer's plans. Inaccurate plans where the draftsman is continually finding errors which must be referred to the architect materially increase the cost of shop drawings."

(*b*) **COST OF MILL DETAILS.**—If material is ordered directly from the rolling mill the price for the necessary cutting to exact length, punching, etc., is based on a standard "card of mill extras."

CARD OF MILL EXTRAS.—If the estimate is to be based on card rates it will be necessary to have the subdivisions a, b, c, d, e, f, r, etc., as follows:

a = 0.15*cts. per lb.* This covers plain punching one size of hole in web only. Plain punching, one size of hole in one or both flanges.

b = 0.25*cts. per lb.* This covers plain punching one size of hole either in web and one flange or web and both flanges. (The holes in the web and flanges must be of same size.)

c = 0.30*cts. per lb.* This covers punching of two sizes of holes in web only. Punching of two sizes of holes either in one or both flanges. One size of hole in one flange and another size of hole in the other flange.

d = 0.35*cts. per lb.* This covers coping, ordinary beveling, riveting or bolting of connection angles and assembling into girders, when the beams forming such girders are held together by separators only.

e = 0.40*cts. per lb.* This covers punching of one size of hole in the web and another size of hole in the flanges.

f = 0.15*cts. per lb.* This covers cutting to length with less variation than $\pm \frac{1}{8}$ in.

r = 0.50*cts. per lb.* This covers beams with cover plates, shelf angles, and ordinary riveted beam work. If this work consists of bending or any unusual work, the beams should not be included in beam classification.

Fittings.—All fittings, whether loose or attached, such as angle connections, bolts, separators, tie rods, etc., whenever they are estimated in connection with beams or channels to be charged at 1.55cts. per lb. over and above the base price. The extra charge for painting is to be added to the price for fittings also. The base price at which fittings are figured is not the base price of the beams to which they are attached but is in all cases the base price of beams 15 in. and under.

The above rates will not include painting, or oiling, which should be charged at the rate of 0.10cts. per lb. for one coat, over and above the base price plus the extra specified above.

For plain punched beams where more than two sizes of holes are used, 0.15cts. per lb. should be added for each additional size of hole, for example, plain punched beams, where three sizes of holes occur would be indicated as: c + 0.15cts., four sizes of holes; e + 0.30cts. For example: a beam with $\frac{5}{8}$ in. and $\frac{3}{4}$ in. holes in the flanges and $\frac{5}{8}$ in. and $\frac{3}{4}$ in. holes in the web should be included in class e.

Cutting to length can be combined with any of the other rates, class d excepted, and would have to be indicated; for example: Plain punching one size of hole in either web and one flange, or web and both flanges, and cutting to length would be marked *bf*, which would establish a total charge of 0.40cts. per lb.

Note to class d.—No extra charge can be added to this class for punching various sizes of holes, or cutting to exact lengths; in other words; if a beam is coped or has connection angles riveted or bolted to it, it makes no difference how many sizes of holes are punched in this beam, the extra will always be the same, namely 0.35cts. When beams have angles or plates riveted to them, and same are not half length of the beam, figure the beams as class d, and the plates and angles as beam connections.

Note to class r.—This rate of 0.50cts. per lb. applies to all the material making up the riveted beam. In case of assembled girders in which one of the beams should be classed as a riveted beam, in making up the estimate, figure only the beam affected as included in class "*r*." When beams have angles or plates riveted to them and same are half length or more than half length of the beam, figure the beams as class "*r*," including the plates or angles and rivets. When 18 in., 20 in., or 24 in. beams are in "*r*" class keep the I's separate from the material (plates, cast iron, separators, angles and rivets) which should go under heading, "15 in. I's and Under."

Beams should be divided as 15 in. I's and under, and 18 in., 20 in. and 24 in. I's. If there are only one or two sizes of beams in any particular class, give exact sizes, instead of "15 in. I's and Under."

In estimating channel roof purlins classify 7 in. channels and smaller as one punched; 8 in. channels and larger as two punched, unless they are shown or noted otherwise, and keep separate from other beams.

No extra charge can be added to curved beams for riveting, cutting to length, etc.

Subdividing work into a large number of classes should be avoided; it is better to have too few classes, rather than too many.

The only subdivision necessary for cast iron columns are: 1 in. and over, and under 1 in. Columns with ornamental work cast on must be kept separate.

Round and Square Bars.—In estimating round and square bars use the standard card for extras, Table III. It is not usual to enforce more than one-half the standard card extras for round and square bars.

Extras.—*Shapes, Plates and Bars:*

(Cutting to length)

Under 3 ft. to 2 ft., inclusive	0.25 ct. per lb.
Under 2 ft. to 1 ft., inclusive	0.50 ct. per lb.
Under 1 ft.	1.55 ct. per lb.

Extras—Plates (*Card of January* 7, 1902):

Base ¼ in. thick, 100 in. wide and under, rectangular (see sketches).

	Per 100 Lb.
Widths—100 in. to 110 in.	$ .05
110 in. to 115 in.	.10
115 in. to 120 in.	.15
120 in. to 125 in.	.25
125 in. to 130 in.	.50
Over 130 in.	1.00
Gages under ¼ in. to and including 3/16 in.	.10
Gages under 3/16 in. to and including No. 8	.15
Gages under No. 8 to and including No. 9	.25
Gages under No. 9 to and including No. 10	.30
Gages under No. 10 to and including No. 12	.40
Complete circles	.20
Boiler and flange steel	.10
Marine and fire box	.20
Ordinary sketches	.10

(Except straight taper plates, varying not more than 4 in. in width at ends, narrowest end not less than 30 in., which can be supplied at base prices.)

TABLE III.

Standard Classification of Extras on Iron and Steel Bars.*

Rounds and Squares.

Squares up to 4½ inches only. Intermediate sizes take the next higher extra.

	Per 100 Lb. Rates.
¾ to 3 in.	
⅝ to 11/16 "	$0.10 extra.
½ to 9/16 "	.20 "
7/16 "	.40 "
⅜ "	.50 "
5/16 "	.70 "
¼ and 9/32 "	1.00 "
7/32 "	2.00 "
3/16 "	2.50 "
3 1/16 to 3½ "	.15 "

* This classification has been quite generally adopted, although several firms issue a special card of extras.

TABLE III.—*Continued.*

STANDARD CLASSIFICATION OF EXTRAS ON IRON AND STEEL BARS.

Flat Bars and Heavy Bands.

$3\frac{9}{16}$ to 4 in.	.25 extra.
$4\frac{1}{16}$ to $4\frac{1}{2}$ "	.30 "
$4\frac{9}{16}$ to 5 "	.40 "
$5\frac{1}{8}$ to $5\frac{1}{2}$ "	.50 "
$5\frac{5}{8}$ to 6 "	.75 "
$6\frac{1}{8}$ to $6\frac{1}{2}$ "	1.00 "
$6\frac{5}{8}$ to $7\frac{1}{2}$ "	1.25 "

Flat Bars and Heavy Bands.

	Per 100 Lb.
1 to 6 in. × $\frac{3}{8}$ to 1 in.	Rates.
1 to 6 " × $\frac{1}{4}$ and $\frac{5}{16}$ "	$0.20 extra.
$\frac{11}{16}$ to $\frac{15}{16}$ " × $\frac{3}{8}$ to $\frac{3}{4}$ "	.40 "
$\frac{11}{16}$ to $\frac{15}{16}$ " × $\frac{1}{4}$ and $\frac{5}{16}$ "	.50 "
$\frac{9}{16}$ and $\frac{5}{8}$ " × $\frac{3}{8}$ to $\frac{1}{2}$ "	.50 "
$\frac{9}{16}$ and $\frac{5}{8}$ " × $\frac{1}{4}$ and $\frac{5}{16}$ "	.70 "
$\frac{1}{2}$ " × $\frac{3}{8}$ and $\frac{7}{16}$ "	.90 "
$\frac{1}{2}$ " × $\frac{1}{4}$ and $\frac{5}{16}$ "	1.10 "
$\frac{7}{16}$ " × $\frac{3}{8}$ "	1.00 "
$\frac{7}{16}$ " × $\frac{1}{4}$ and $\frac{5}{16}$ "	1.20 "
$\frac{3}{8}$ " × $\frac{1}{4}$ and $\frac{5}{16}$ "	1.50 "
$1\frac{1}{8}$ to 6 in. × $1\frac{1}{16}$ to $1\frac{3}{16}$ in.	.10 "
$1\frac{1}{8}$ to 6 " × $1\frac{1}{4}$ to $1\frac{1}{2}$ "	.20 "
$1\frac{3}{4}$ to 6 " × $1\frac{5}{8}$ to $2\frac{3}{4}$ "	.30 "
$3\frac{1}{8}$ to 6 " × 3 to 4 "	.40 "

Light Bars and Bands.

	Per 100 Lb.
$1\frac{1}{2}$ to 6 in. × Nos. 7, 8, 9 and $\frac{3}{16}$ in.	$0.40 extra.
$1\frac{1}{2}$ to 6 in. × Nos. 10, 11, 12 and $\frac{1}{8}$ in.	.60 "
1 to $1\frac{7}{16}$ in. × Nos. 7, 8, 9 and $\frac{3}{16}$ in.	.50 "
1 to $1\frac{7}{16}$ in. × Nos. 10, 11, 12 and $\frac{1}{8}$ in.	.70 "
$\frac{13}{16}$ to $\frac{15}{16}$ in. × Nos. 7, 8, 9 and $\frac{3}{16}$ in.	.70 "
$\frac{13}{16}$ and $\frac{15}{16}$ in. × Nos. 10, 11, 12 and $\frac{1}{8}$ in.	.80 "
$\frac{11}{16}$ and $\frac{3}{4}$ in. × Nos. 7, 8, 9 and $\frac{3}{16}$ in.	1.00 "
$\frac{11}{16}$ and $\frac{3}{4}$ in. × Nos. 10, 11, 12 and $\frac{1}{4}$ in.	1.20 "
$\frac{9}{16}$ and $\frac{5}{8}$ in. × Nos. 7, 8, 9 and $\frac{3}{16}$ in.	1.20 "
$\frac{9}{16}$ and $\frac{5}{8}$ in. × Nos. 10, 11, 12 and $\frac{1}{8}$ in.	1.30 "
$\frac{1}{2}$ × Nos. 7, 8, 9 and $\frac{3}{16}$ in.	1.30 "
$\frac{1}{2}$ × Nos. 10, 11, 12 and $\frac{1}{8}$ in.	1.50 "
$\frac{7}{16}$ × Nos. 7, 8, 9 and $\frac{3}{16}$ in.	1.80 "
$\frac{7}{16}$ × Nos. 10, 11, 12 and $\frac{1}{8}$ in.	2.10 "
$\frac{3}{8}$ × Nos. 7, 8, 9 and $\frac{3}{16}$ in.	1.90 "
$\frac{3}{8}$ × Nos. 10, 11, 12 and $\frac{1}{8}$ in.	2.40 "

Mill Orders.—In mill orders the following items should be borne in mind. Where beams butt at each end against some other member, order the beams $\frac{1}{2}$ in. shorter than the figured lengths this will allow a clearance of $\frac{1}{4}$ in. if all beams come $\frac{3}{8}$ in. too long. Where beams are to be built into the wall, order them in full lengths, making no allowance for clearance. Order small plates in multiple lengths. Irregular plates on which there will be considerable waste should be ordered cut to templet. Mills will not make reentrant cuts in plates. Allow $\frac{1}{4}$ in. for each milling for members that have to be faced. Order web plates for girders $\frac{1}{4}$ to $\frac{1}{2}$ in. narrower than the distance back to back of angles. Order as nearly as possible every thing cut to required length, except where there is liable to be changes made, in which case order long lengths.

It is often possible to reduce the cost of mill details by having the mills do only part of the work, the rest being done in the field, or by sending out from the shop to be riveted on in the field connection angles and other small details that would cause the work to take a very much higher

price. Standard connections should be used wherever possible, and special work should be avoided.—For additional notes on ordering material, see Chapter XII.

In estimating the cost of plain material in a finished structure the shipping weight from the structural shop is wanted. The cost of material f. o. b. the shop must therefore include the cost of waste, paint material, and the freight from the mill to the shop. The waste is variable but as an average may be taken at 4 per cent. Paint material may be taken as two dollars per ton. The cost of plain material at the shop would be

Average cost per lb. f. o. b. mill, say	1.75 cts.
Add 4 per cent for waste	.07 "
Add $2.00 per ton for paint material	.10 "
Add freight from mill to shop (Pittsburg to St. Louis)	.225 "
Total cost per pound f. o. b. shop	2.145"

To obtain the average cost of steel per pound multiply the pound price of each kind of material by the percentage that this kind of material is of the whole weight, the sum of the products will be the average pound price.

(*c*) **COST OF SHOP LABOR.**—The cost of shop labor may be calculated for the different parts of the structure, or may be calculated for the structure as a whole. The following costs are based on an average charge of 40 cents per hour and include detailing and shop labor. The cost of fabricating beams, channels and angles which are simply punched or have connection angles loose or attached should be estimated on the basis of mill details, which see.

SHOP COSTS OF STEEL FRAME BUILDINGS.—The following costs of different parts of steel frame office and mill structures are a fair average.

Columns.—In lots of at least six, the shop cost of columns is about as follows: Columns made of two channels and two plates, or two channels laced cost about 0.80 to 0.70 cts. per lb., for columns weighing from 600 to 1,000 lb. each; columns made of 4 angles laced cost from 0.80 to 1.10 cts. per lb.; columns made of two channels and one I-beam, or three channels cost from 0.65 to 0.90 cts. per lb.; columns made of single I-beams, or single angles cost about 0.50 cts. per lb.; and Z-bar columns cost from 0.70 to 0.90 cts. per lb.

Plain cast columns cost from 1.50 to 0.75 cts. per lb., for columns weighing from 500 to 2,500 lb., and in lots of at least six.

Roof Trusses.—In lots of at least six, the shop cost of ordinary riveted roof trusses in which the ends of the members are cut off at right angles is about as follows: Trusses weighing 1,000 lb. each, 1.15 to 1.25 cts. per lb.; trusses weighing 1,500 lb. each, 0.90 to 1.00 cts. per lb.; trusses weighing 2,500 lb. each, 0.75 to 0.85 cts. per lb.; and trusses weighing 3,500 to 7,500 lb. 0.60 to 0.75 cts. per lb. Pin-connected trusses cost from 0.10 to 0.20 cts. per lb. more than riveted trusses.

Eave Struts.—Ordinary eave struts made of 4 angles laced, whose length does not exceed 20 to 30 ft., cost for shop work from 0.80 to 1.00 cts. per lb.

Plate Girders.—The shop work on plate girders for crane girders and floors will cost from 0.60 to 1.25 cts. per lb., depending upon the weight, details and number made at one time.

TABLE IV.

SHOP COST OF CIRCULAR AND RECTANGULAR BINS AND STAND-PIPES, NOT INCLUDING HOPPERS OR BOTTOMS.

Thickness of Metal, In.	Shop Cost in Cents per Lb.	
	Water Tight.	Bins.
1/4	0.90	0.80
5/16	0.85	0.75
3/8	0.80	0.70
1/2	0.75	0.65

SHOP COSTS OF BINS AND STAND-PIPES.*—Shop costs for circular and rectangular bins and stand-pipes are given in Table IV, while shop costs for bin and elevated tank bottoms are given in Table V. The shop cost of towers for elevated tanks are given in Table VI.

TABLE V.

SHOP COST OF BOTTOMS FOR CIRCULAR AND RECTANGULAR BINS AND STAND-PIPES.

Thickness of Material, In.	Flat Bottom, Cents per Lb.	Spherical Bottom, Cents per Lb.	Conical Bottom, Cents per Lb.	Hopper Bottom, Cents per Lb.
$\frac{1}{4}$	1.50	4.00	3.50	2.50
$\frac{5}{16}$	1.45	4.15	3.00	2.40
$\frac{3}{8}$	1.40	4.40	2.75	2.25
$\frac{1}{2}$	1 25	4.50	2.50	2.00

TABLE VI.

SHOP COST OF TOWERS FOR ELEVATED TANKS AND BINS.

Weight of Tower and Bracing in Lb.	Shop Cost in Cents per Lb.	
	Adjustable Bracing.	Riveted Bracing.
10,000 and less	1.30	1.20
10,000 to 20,000	1.25	1.10
20,000 to 50,000	1.15	1.05
50,000 and up	1.10	1.00

SHOP COSTS OF INDIVIDUAL PARTS OF BRIDGES.*—The cost of fabricating joists and other similar members should be estimated on the basis of mill details, which see.

Eye-Bars.—The shop cost of eye-bars varies with the size and length of the bars and the number made alike. The following costs are a fair average: Average shop costs of bars 3 in. and less in width and $\frac{3}{4}$ in. and less in thickness is from 1.20 to 1.80 cts. per lb., depending upon the length and size. A good order of bars running $2\frac{1}{2}$ in. × $\frac{3}{4}$ in. to 3 in. × $\frac{3}{4}$ in., and from 16 to 20 ft. long, with few variations in size, will cost about 1.20 cts. per lb. Large bars in long lengths ordered in large quantities can be fabricated at from 0.55 to 0.75 cts. per lb. To get the total cost of eye-bars the cost of bar steel must be added to the shop cost. Half card extras given in Table III should ordinarily be added to the base price of plain steel bars.

Chords, Posts and Towers.—In lots of at least four, the shop cost is about as follows: Members made of two channels and a top cover plate with lacing on the bottom side, or two channels laced on both sides cost about 1.00 to 0.85 cts. per lb. for pin-connected members weighing from 600 to 1,500 lb.; and about 0.80 to 0.70 cts. per lb. for members with riveted end connections. Members made of four angles laced cost from 0.80 to 1.10 cts. per lb. for members with riveted ends. Members made of two angles battened will cost about 0.50 cts. per lb. Angles used without end connections should have their cost estimated on the basis of mill details, which see.

Pins.—The cost of chord pins will vary with the size, number and other requirements. The shop cost of chord pins and nuts may be estimated at from 2.00 to 3.00 cts. per lb. Rollers will cost practically the same as pins. Rolled rounds (pin rounds) are used for making pins and rollers.

Latticed Fence.—The shop cost of light simple latticed fence made of two 2 in. × 2 in. angles, with double lacing and about 18 in. deep, will be about 2.00 cts. per lb.; while the shop cost of latticed fence, with ornamental rosettes or ornamental plates, may be as much as 4.00 to 5.00 cts. per lb.

Floorbeams and Stringers.—Plate girders used for floorbeams and stringers will cost from 0.60 to 1.25 cts. per lb. depending upon the weight, details and number made at one time. Floorbeams made of rolled I-beams will cost from 0.50 to 0.75 cts. per lb.

* Prewar, 1914, costs are given.

SHOP COSTS OF BRIDGES AS A WHOLE.*—The cost will be taken up under the head of pin-connected bridges, riveted bridges, plate girder bridges, combination bridge metal, and Howe truss metal.

Shop Costs of Pin-connected Bridges.—The shop costs of pin-connected highway or railway bridges, exclusive of fence and joists, are about as follows:

Bridges weighing	5,000 lb. and less	1.30 cts. per lb.
" "	5,000 to 10,000 lb.	1.20 " " "
" "	10,000 to 20,000 lb.	1.00 " " "
" "	20,000 to 40,000 lb.	0.90 " " "
" "	40,000 to 60,000 lb.	0.80 " " "
" "	60,000 to 100,000 lb.	0.75 " " "
" "	100,000 to 150,000 lb.	0.70 " " "
" "	150,000 and up	0.65 " " "

These costs include detailing and one coat of shop paint. For reaming add 0.15 cts. per lb.

Shop Costs of Riveted Truss Bridges.—The shop costs of riveted truss highway or railway bridges, exclusive of fence and joists, are about as follows:

Bridges weighing	5,000 lb. and less	1.15 cts. per lb.
" "	5,000 to 10,000 lb.	1.00 " " "
" "	10,000 to 20,000 lb.	0.90 " " "
" "	20,000 to 40,000 lb.	0.85 " " "
" "	40,000 to 60,000 lb.	0.75 " " "
" "	60,000 to 100,000 lb.	0.70 " " "
" "	100,000 to 150,000 lb.	0.65 " " "
" "	150,000 lb. and up	0.60 " " "

These costs include detailing and one coat of shop paint. For reaming add 0.15 cts. per lb.

Shop Costs of Plate Girder Bridges.—The shop costs of plate girder highway or railway bridges, exclusive of fence and joists, are about as follows:

Spans weighing	10,000 lb. and less	0.90 cts. per lb.
" "	10,000 to 20,000 lb.	0.85 " " "
" "	20,000 to 40,000 lb.	0.75 " " "
" "	40,000 to 60,000 lb.	0.70 " " "
" "	60,000 to 100,000 lb.	0.60 " " "
" "	100,000 and up	0.50 " " "

These costs include detailing and one coat of shop paint. For reaming add 0.15 cts. per lb.

Shop Costs of Tubular Piers and Culverts.—The shop costs of steel tubular pier shells and steel culvert pipe are about as follows:

Tubes	18 in. to 24 in. diameter,	$\frac{1}{4}$ in. metal	1.00 cts. per lb.
"	24 in. to 30 in. diameter,	$\frac{1}{4}$ in. to $\frac{3}{8}$ in. metal	0.75 to 0.65 " " "
"	30 in. to 48 in. diameter,	$\frac{1}{4}$ in. to $\frac{3}{8}$ in. metal	0.70 to 0.60 " " "
"	48 in. to 72 in. diameter,	$\frac{1}{4}$ in. to $\frac{1}{2}$ in. metal	0.65 to 0.50 " " "
"	72 in. and up	$\frac{3}{8}$ in. to $\frac{5}{8}$ in. metal	0.50 to 0.45 " " "

The above shop costs include detailing and one coat of shop paint. The necessary bracing and rods for tubular piers are included.

Shop Cost of Combination Bridge Metal.—Where the bars and rods are standard and the castings are made from standard patterns, the metal for combination bridges can be fabricated at about the same cost per pound as for pin-connected spans weighing the same as the weight of the metal in the combination bridges.

* **Prewar, 1914, costs are given.**

Shop Cost of Howe Truss Bridge Metal.—The shop cost of highway bridge castings made from standard patterns, is from 1.50 to 2.00 cts. per lb. The shop costs of the plates, rods and other miscellaneous iron work will be from 2.00 to 2.50 cts. per lb.

COST OF ERECTION OF STEEL FRAME OFFICE AND MILL BUILDINGS AND MINE STRUCTURES.—In estimating the cost of erection of structural steel work it is best to divide the cost into (*a*) cost of placing and bolting steel, and (*b*) cost of riveting. The cost will be based on labor at an average price of $3.20 per day of 8 hours or 40 cts. per hour.

(*a*) **Cost of Placing and Bolting.**—The cost of placing and bolting mill buildings for ordinary conditions may be estimated at from $6.00 to $8.00 per ton. The cost of placing and bolting up steel office buildings may be estimated at from $5.00 to $9.00 per ton. The cost of placing and bolting up steel bins may be estimated at from $10.00 to $15.00 per ton. The cost of placing and bolting up head frames may be estimated at from $12.00 to $18.00 per ton.

(*b*) **Cost of Riveting.**—It will cost from 6 to 10 cts. per rivet to drive $\frac{5}{8}$ or $\frac{3}{4}$ in. rivets by hand in structural framework where a few rivets are found in one place. A fair average is 7 cts. per rivet. The same size rivets can be driven in tank work for from 4 to 7 cts. per rivet, with 5 cts. per rivet as a fair average.

The cost of riveting by hand is distributed about as follows:

3 men, 2 driving and 1 bucking up, at $3.50 per day of 8 hours	$10.50
1 rivet heater at $3.00 per day of 8 hours	3.00
Coal, tools, superintendence	1.50
Total per day	$15.00

On structural work a fair day's work driving $\frac{3}{4}$ in. or $\frac{5}{8}$ in. rivets will be from 150 to 250, depending upon the amount of scaffolding required. This makes the total cost from 6 to 10 cts. per rivet.

On bin work when the rivets are close together and little staging is required the gang above will drive from 200 to 400 rivets per day. This makes the total cost from about 4 to 7 cts. per rivet.

Rivets can be driven by power riveters for one-half to three-fourths the above, not counting the cost of installation and air. The added cost for power and equipment makes the cost of driving field rivets with pneumatic riveters about the same as the cost of driving field rivets by hand.

Soft iron rivets $\frac{1}{2}$ in. and under can be driven cold for about one-half what the same rivets can be driven hot, or even less.

Cost of Erection.—Small steel frame buildings will cost about $10.00 per ton for the erection of the steel framework, if trusses are riveted and all other connections are bolted. The cost of laying corrugated steel is about $0.75 per square when laid on plank sheathing, $1.25 per square when laid directly on the purlins, and $2.00 per square when laid with anti-condensation lining. The erection of corrugated steel siding costs from $0.75 to $1.00 per square. The cost of erecting heavy machine shops, all material riveted and including the cost of painting but not the cost of the paint, is about $8.50 to $9.00 per ton. Small buildings in which all connections are bolted may be erected for from $5.00 to $6.00 per ton. The cost of erecting the structural framework for office buildings will vary from $6.00 to $10.00 per ton.

Actual Costs of Erection.—The cost of erecting the East Helena transformer building, 1897, was $12.80 per ton, including the erection of the corrugated steel and transportation of the men. The cost of erecting the Carbon Tipple was $8.80 per ton, including corrugated steel. The cost of erection of the Basin & Bay State Smelter was $8.20 per ton, including the hoppers and corrugated steel.

The cost of erecting the structural steel work for the Great Northern Ry. Grain Elevator, Superior, Wisconsin, was $13.25 per ton including the driving of all rivets. There were 10,600 tons of structural steel work, and 2,000,000 field rivets, or nearly 200 field rivets per ton of structural steel.

Erection of Structural Steel for an Armory.*—The structural framework for the new armory of the University of Illinois, consists of three-hinged arches having a span of 206 ft., and a center height of 94 ft. 3 in. The arches are spaced 26 ft. 6 in. centers and are braced in pairs. The total weight of structural steel was 985 tons, and contained 15,400, $\frac{7}{8}$ in. and 14,900, $\frac{3}{4}$ in. or a total of 30,300 field rivets. The cost of erecting the structural steel, including field riveting was $9.55 per ton. The average cost of driving the field rivets was 13.1 cts. each.

COST OF ERECTION OF STEEL BRIDGES.—The cost of erection ordinarily includes (1) the cost of hauling the bridge to the bridge site; (2) the building of the falsework and the placing of the steel in position; (3) the riveting up of the bridge, and (4) painting the steel and the woodwork.

Hauling.—Transportation over country roads will ordinarily cost about 25 cts. per ton-mile, in addition to the cost of loading and unloading. In estimating the cost of hauling on any particular job the length of haul, kind of roads, price of teams and labor, and the character of the teams should be considered. The cost of loading on the wagons and unloading will depend upon the local conditions, but will ordinarily be from 25 to 50 cts. per ton. For railroad bridges the steel work may ordinarily be brought directly to the site by rail.

Falsework.—If piles are to be used the cost should be carefully estimated. The cost of the piles in place will vary with the cost of piles and local conditions. Under ordinary conditions piles in falsework will cost from 25 to 50 cts. per lineal foot in place. The cost of the timber will depend upon local conditions and upon what use is made of it after erection. The flooring plank in highway bridges, and ties and guard timbers in railway bridges can often be used in the falsework without serious injury. The cost of erecting the timber in the falsework will ordinarily be from $6.00 to $8.00 per thousand ft. B. M.

Erection of Tubular Piers.—The cost of setting tubular piers for highway bridges will depend upon the conditions. Tubes 36 in. in diameter and 20 ft. long have been set in favorable locations for $25.00 per pair, not including the driving of the piles or the placing of the concrete. It is, however, not safe to estimate the cost of setting tubes from 36 to 48 in. in diameter under even favorable conditions at less than $2.00 per lineal foot of tube. When the cost of setting tubes is estimated by weight, it should be figured at from $15.00 to $20.00 per ton, for ordinary conditions. It will commonly cost from 25 to 50 cts. per lineal ft to drive piles in tubes, in addition to the cost of the piles, which will vary from 10 to 20 cts. per lineal foot. The concrete will commonly cost from $6.00 to $8.00 per cu. yd. in place in the tube.

Placing and Bolting.—The cost of placing and bolting up riveted highway spans, and erecting pin-connected highway spans, no rivets being driven, is about as follows:

Highway spans from	30 to 60 ft.	$12.00 to $15.00 per ton.
" " "	60 to 100 ft.	10.00 to 12.00 " "
" " "	100 to 150 ft.	9.00 to 10.00 " "
" " "	150 ft. and up	8.00 " "

The cost of placing and bolting up railroad spans will depend so much upon the local conditions and equipment that it is difficult to give general costs.

The cost of driving field rivets in pin-connected spans will vary from 7 to 12 cts. per rivet while the cost of driving field rivets in riveted trusses will vary from 6 to 10 cts per rivet. The number of rivets in riveted low truss highway bridges depends upon the number of panels and the style of details, and will be about 155 to 200 for a three-panel bridge, and 400 to 500 for a six-panel bridge. The number of rivets in through riveted highway bridges will be about 250 to 300 for a four-panel bridge, and 1,300 to 1,500 for a nine-panel bridge. Pin-connected bridges ordinarily have about $\frac{1}{3}$ to $\frac{1}{2}$ as many field rivets as a riveted bridge of similar dimensions.

The approximate number of field rivets in single track railway bridges, designed for E 55 loading, are given in Table VII.

* Engineering and Contracting, Aug. 6, 1913.

TABLE VII.

NUMBER OF FIELD RIVETS IN RAILWAY BRIDGES, SINGLE TRACK, E 55 LOADING. (HARRIMAN LINES.)

Plate Girders.				Through Truss Bridges.			
Deck.		Through.		Riveted.		Pin-Connected.	
Span, Ft.	Number of Field Rivets.	Span, Ft.	Number of Field Rivets.	Span, Ft.	Number of Field Rivets.	Span, Ft.	Number of Field Rivets.
30	100	30	600	100	2,900	150	2,800
40	200	40	1,200	110	2,900	160	3,000
50	300	50	1,300	125	4,300	180	3,200
60	400	60	1,700	140	5,300	200	3,200
70	500	70	1,900	150	5,600	....	....
80	500	80	2,000	....	....	....	....
90	500	90	2,200	....	....	....	....
100	600	100	2,400	....	....	....	....

The field rivets on the 20th St. Viaduct, Denver, Colorado, cost 7 cts. each. The rivets were driven by air riveters.

Actual Costs of Erecting Railway Bridges.—The cost of erecting railway bridges on the A. T. & S. F. Ry. in 1907 are given in the report of the Assoc. of Ry. Supt. of B. & B. as follows:—

Trusses, 984 tons erected, cost $4.63 per ton.

Plate Girders, 2,784 tons erected, cost $5.49 per ton.

I-Beams, 2,837 tons erected, cost $2.88 per ton.

All girders and I-beams were erected with a steam wrecker and the through spans with a derrick car. The reason for the plate girders costing more to erect than the through trusses was that many of the plate girders were on second track where the old girders had to be cut apart and moved to the outside and heavier girders put in their place. All rivets were driven by hand. For additional examples of actual costs, see Gillette's "Cost Data."

Transportation.—Fabricated structural steel commonly takes a "fifth-class rate" when shipped in car load lots, and a "fourth-class rate" when shipped "local" (in less than car load lots). The minimum car load depends upon the railroad and varies from 20,000 to 30,000 lb. Tariff sheets giving railroad rates may be obtained from any railroad company. The shipping clerk should be provided with the clearances of all tunnels and bridges on different lines so that the car may be properly loaded.

Freight Rates.—The freight rates (1924) on finished steel products in car load shipments from the Pittsburgh District, including plate, structural shapes, merchant steel and iron bars, pipe fittings, plain and galvanized wire, nails, rivets, spikes and bolts (in kegs), black sheets (except planished), chain, etc., are as follows, in cts. per 100 lb. in carload shipments; Baltimore, 31; Birmingham, 58; Boston, 36½; Buffalo, 26½; Chicago, 34; Cincinnati, 29; Cleveland, 21½; Denver, 126; Detroit, 29; Kansas City, 73½; New Orleans, 67; New York, 34; Pacific Coast (all rail), 115; Philadelphia, 32; St. Louis, 43; St. Paul, 60.

COST OF PAINTING.—The amount of materials required to make a gallon of paint and the surface of steel work covered by one gallon are given in Table VIII. Structural steel should be painted with one coat of linseed oil, linseed oil with lamp-black filler, or red lead paint at the shop; and two coats of first-class paint after erection. The two field coats should be of different colors; care being used to see that first coat is thoroughly dry before applying the second coat. Steel bridges and exposed steel frame buildings ordinarily require repainting every three or four years.

The steel work in the extension to the 16th St. Viaduct, Denver, Colo., was painted with red lead paint mixed in the following proportions,—100 lb. red lead, 2 lb. lamp-black and 4.125 gallons

of linseed oil. This mixture made 6 gallons of mixed paint of a chocolate color, and gave 1.455 gallons of paint for each gallon of oil.

TABLE VIII.

AVERAGE SURFACE COVERED PER GALLON OF PAINT.
PENCOYD HAND BOOK.

Paint.	Volume of Oil.	Pounds of Pigment.	Volume and Weight of Paint.		Square Feet.	
			Gal.	Lb.	1 Coat.	2 Coats.
Iron oxide (powdered)	1 gal.	8.00	1.2 =	16.00	600	350
Iron oxide (ground in oil)	1 gal.	24.75	2.6 =	32.75	630	375
Red lead (powdered)	1 gal.	22.40	1.4 =	30.40	630	375
White lead (ground in oil)	1 gal.	25.00	1.7 =	33.00	500	300
Graphite (ground in oil)	1 gal.	12.50	2.0 =	20.50	630	350
Black asphalt	1 gal. (turp.)	17.50	4.0 =	30.00	515	310
Linseed oil (no pigment)	1 gal.	......	......	......	875	......

Light structural work will average about 250 sq. ft., and heavy structural work about 150 sq. ft. of surface per net ton of metal, while No. 20 corrugated steel has 2,400 sq. ft. of surface.

It is the common practice to estimate ½ gallon of paint for the first coat and ⅜ gallon for the second coat per ton of structural steel, for average conditions.

The price of paint materials in small quantities in Chicago are (1914) about as follows: Linseed oil, 50 to 60 cts. per gal.; iron oxide, 1 to 2 cts. per lb.; red lead, 7 to 8 cts. per lb.; white lead, 6 to 7 cts. per lb.; graphite, 6 to 10 cts. per lb.

A good painter should paint 1,200 to 1,500 sq. ft. of plate surface or corrugated steel or 300 to 500 sq. ft. of structural steel work in a day of 8 hours; the amount covered depending upon the amount of staging and the paint. A thick red lead paint mixed with 30 lb. of lead to the gallon of oil will take fully twice as long to apply as a graphite paint or linseed oil. The cost of applying paint is roughly equal to the cost of a good quality of paint, the cost per ton depending on the spreading qualities of the paint. This rule makes the cost of applying a red lead paint with 30 lb. of pigment per gallon of oil from two to three times the cost of applying a good graphite paint, per ton of structural steel. For additional data on paints, see Chapter XV.

MISCELLANEOUS COSTS.*—The following approximate costs will be of value in making preliminary estimates. The cost of construction depends so much upon local conditions that average costs should only be used as a guide to the judgment of the engineer.

MILL BUILDING FLOORS.—The following costs are for floors resting on a good compact soil and do not include unusual difficulties.

Timber Floor on Pitch-Concrete Base.—The cost varies from about $1.25 per sq. yd. for a 2-in. pine sub-floor and a ⅞-in. pine finish, to about $1.75 per sq. yd. for a 2-in. pine sub-floor and a ⅞-in. maple finish.

Concrete Floor on Gravel Sub-base.—The cost varies from $1.25 to $2.00 per sq. yd.

Creosoted Timber Block Floor.—Creosoted timber blocks 3 in. to 4 in. thick, laid on a 6-in. concrete base, will cost from $2.50 to $3.50 per sq. yd.

ROOFING FOR MILL BUILDINGS.—The following costs include the cost of materials and the cost of laying, but do not include the cost of the sheathing.

Corrugated Steel Roofing.—The weight of corrugated steel roofing and siding may be obtained from Table I, Chapter I. The price of corrugated steel may be obtained from current quotations in Engineering News or Iron Age. The cost of laying corrugated steel is about $0.75 per square when laid on plank sheathing, $1.25 per square when laid directly on the purlins, and $2.00 per square when laid with anti-condensation lining. The erection of corrugated siding costs from $0.75 to $1.00 per square. Asbestos paper costs from 3½ to 4 cts. per lb. Galvanized

* Prewar, 1914, costs are given.

wire netting, No. 19, costs 25 to 30 cts. per square of 100 sq. ft. Brass wire, No. 20, costs about 20 cts. per lb. No. 9 galvanized wire costs about 3 cts. per lb. For trimmings, flashing, ridge roll, etc., add 1 ct. per lb. to the base price of corrugated steel.

Tar and Gravel Roofing.—Four- or five-ply tar and gravel roofing, for average conditions, costs from $3.75 to $4.00 per square, not including sheathing. Five hundred squares of 5-ply tar and gravel roofing, in 1912, in the middle west, cost $3.93 per square, not including sheathing.

Tin Roofing.—Tin roofing costs from $7.00 to $9.00 per square, not including sheathing.

Slate Roofing.—Slate roofing costs from $7.00 to $12.00 per square, not including sheathing.

Tile Roofing.—The cost of tile roofing is variable, depending upon style of roof and location and local conditions, and may vary from $13.00 to $30.00 per square, not including sheathing.

WINDOWS.—Windows with wooden frames and sash, and double strength glass, will cost from 25 to 50 cts. per sq. ft. of opening. Windows with metal frames and sash and wire glass, will cost from 45 to 55 cts. per sq. ft. of opening.

SKYLIGHTS.—Skylights with metal frames and sash and wire glass, will cost from 50 to 60 cts. per sq. ft. Skylights made of translucent fabric stretched on wooden frames, will cost from 25 to 30 cts. per sq. ft. Louvres without frames, will cost about 25 cts. per sq. ft.

CIRCULAR VENTILATORS.—Circular ventilators will cost about as follows:—12-in., $2.00; 18-in., $6.75; 24-in., $10.00; 36-in., $15.00 each, when ordered in lots of at least six.

ROLLING STEEL SHUTTERS.—Rolling steel shutters will cost $0.75 to $1.00 per sq. ft.

WATERPROOFING.—The following costs for waterproofing engineering structures are taken from the Proceedings of the American Railway Engineering Association, Vol. 12, 1911. (1) Bridge floor, 6-ply felt and pitch, 12½ cts. per sq. ft., including protection over waterproofing. (2) Trough bridge floor, 4-ply burlap and asphalt, 10 to 16½ cts. per sq. ft. (3) Bridge floor, 3-ply burlap and asphalt, and asphalt mastic, 16 cts. per sq. ft. (4) Concrete slab bridge floor, 5-ply felt, 1-ply burlap and pitch, 15½ cts. per sq. ft., including a 10 year guarantee.

MISCELLANEOUS MATERIALS.—The following prices are for small lots, f.o.b. Pittsburgh (May, 1914).

Chain.—Standard chain, $\frac{3}{16}$ in., 7½ cts. per lb.; ½ in., 3 cts. per lb.; 1 in., 2.6 cts. per lb. For BB chain, add 1½ cts. per lb., and for BBB chain, add 2 cts. per lb.

Nails.—Base price of nails, $2.00 per keg of 100 lb.—20d to 60 d nails are base; for 10d to 16d, add 5 cts. per keg; for 8d and 9d, add 10 cts. per keg; for 6d and 7d, add 20 cts. per keg; for 4d and 5d, add 30 cts. per keg; for 3d, add 45 cts. per keg, and for 2d, add 70 cts. per keg.

Gas Pipe.—Gas pipe costs about as follows:—Standard gas pipe 1 in. diam., black, 3½ cts. per ft., glavanized, 5 cts. per ft.; 2 in. diam., black, 7½ cts. per ft., galvanized, 11 cts. per ft.; 3 in. diam., black, 16½ cts. per ft., galvanized, 23 cts. per ft.

Steel Railroad Rails.—Bessemer rails, $28 per gross ton (2240 lb.); open-hearth, $30 per gross ton.

Wire Rope.—The cost of steel wire rope is about as follows:—⅝ in. rope, 10 cts. per lineal ft.; ¾ in. rope, 13 cts. per lineal ft.; 1 in. rope, 20 cts. per lineal ft.; 1½ in. rope, 45 cts. per lineal ft.

Manila Rope.—Manila rope costs about 12½ cts. per lb. Sisal rope costs about 9 cts. per lb.

HARDWARE AND MACHINISTS SUPPLIES.—Prices of hardware and machinists supplies are for the most part quoted by giving a discount from standard list prices. The "Iron Age Standard Hardware Lists," price $5.00, may be obtained from the Iron Age Book Department, 239 W. 39th St., New York Discounts from these standard lists are given each week in Iron Age The base prices of structural materials are given in the first issue of each month of Engineering News-Record, and are given in each issue of Iron Age.

REFERENCES.—For detailed estimates of steel mill buildings and additional data on the cost of steel mill buildings see the authors' "The Design of Steel Mill Buildings." For detailed estimates of steel highway bridges and additional data on the cost of steel highway bridges, see the author's "The Design of Highway Bridges." For data on the cost of retaining walls, bins and grain elevators, see the author's "The Design of Walls, Bins and Grain Elevators." For data on the cost of steel head frames, coal tipples, and other mine structures, see the author's "The Design of Mine Structures."

CHAPTER XIV.

Erection of Structural Steel.

METHODS OF ERECTION.—The method used in erecting a steel structure will depend upon the type of structure, the size of the structure, the risk to be taken, as in bridge erection, whether the structure is to be erected without interfering with traffic, as in erecting a railroad bridge to replace an existing structure, or in erecting a building over furnaces or working machinery, the available tools, and local conditions. The tendency of modern structural steel erection practice is, as far as possible, to use derrick cars for erecting railway bridges and locomotive cranes for erecting mill buildings and other structures.

The methods of erection that may be used for erecting different steel structures are as follows.

Plate Girders and Short Riveted Spans.—Plate girders up to about 60 ft. span are very commonly riveted up complete with cross frames and bracing, either at the shop or at the site, and are placed in position on the abutments. With plate girders longer than 60 ft. and short riveted trusses one girder or truss is placed in position at a time and the floorbeams and bracing are put in place after the girders or trusses are in place. The girders or trusses may be swung into place by a stiff-leg derrick or a guy derrick set up alongside the track or back of the abutment where there is no track; by a derrick car, or may be hoisted into place by a gin pole. Where falsework has been placed girders are picked up from the cars by two gallows frames, one near each end of the span, or by one gallows frame and a derrick. Plate girders may also be put in place by sliding into place either longitudinally or transversely, or by jacking and cribbing.

Truss Bridges.—Riveted trusses up to a span of 100 to 125 ft. may be riveted up on the bank and be swung into place by a boom traveler or a derrick. The floorbeams and bracing are then put in place and the span riveted up. Where falsework is required the bridge may be erected by a gantry or outside traveler placed outside of the trusses, by a boom traveler running on a track placed inside the trusses, or by a derrick car. The gantry or outside traveler is commonly used for long spans and for highway spans where no tracks are available. The boom traveler is commonly used for elevated railway and highway viaducts. The derrick car is now commonly used for erecting railway bridges and is sometimes used for erecting viaducts.

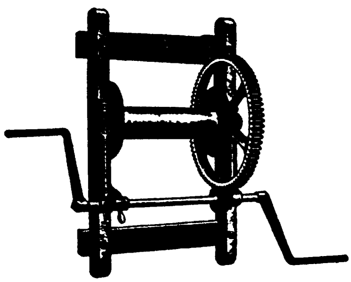
Cantilever Bridges.—Cantilever bridges are commonly erected by means of an overhang traveler running on the completed portion, the structure being built out from the shore. Cantilever bridges are sometimes erected on falsework in the same manner as simple trusses.

Arch Bridges.—Arches may be erected on falsework in the same manner as simple truss spans, or may be cantilevered out from each abutment, the cantilever being supported by temporary cables running over a tower placed back of the abutments.

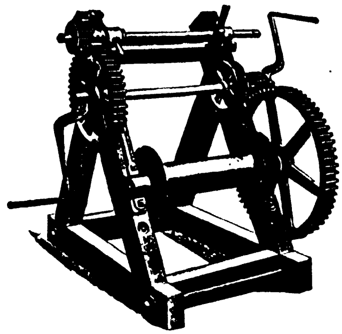
High Viaducts.—High steel viaducts are commonly erected by means of an overhang or boom traveler running on a track on top of the viaduct girders. The overhang or boom is long enough to place a tower in advance with the traveler on the completed portion. Derrick cars have also been used for erecting high steel viaducts. The towers and the girders may be erected by means of gin poles. The tower bents may be bolted up before raising or may be erected and bolted up in place.

Roof Trusses, Mill and Office Buildings.—Where there is sufficient room, roof trusses up to 150 ft. span may be riveted or bolted up on the ground and may then be raised into position by means of one or two gin poles. Two gin poles should be used for long trusses. Care should be used not to cripple the lower chord. With light trusses, the lower chord members should be stiffened by means of timbers or other stiff members temporarily bolted or lashed to the member. Columns and beams in office buildings may be erected with stiff-leg or guy derricks, or "A"

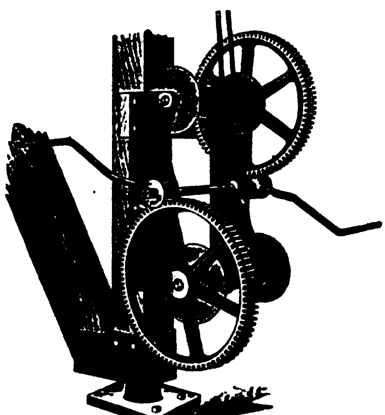
derricks may be used for loads up to 5 tons. The bents of steel mill buildings may be erected in the same manner. Roof arches and train sheds are sometimes erected by means of falsework, which is moved as the erection proceeds. Boom-tower derricks running on tracks are found



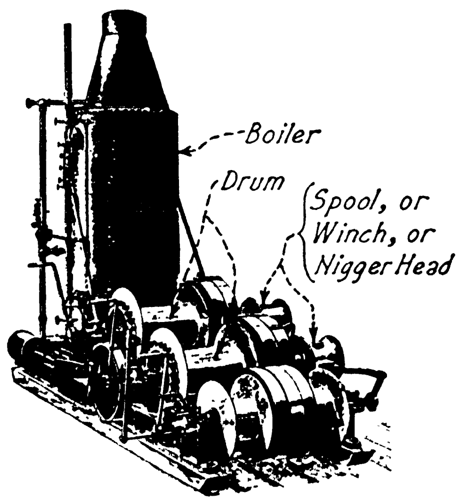
(a) CRAB



(b) WINCH



(c) DERRICK CRAB



(d) HOISTING ENGINE

FIG. 1. HOISTS FOR STEEL ERECTION.

very convenient. Locomotive cranes are now used for erecting mill buildings and similar structures where tracks are available.

Elevated Towers and Tanks.—The towers for high tanks are commonly erected by means of a gin pole. A gin pole long enough to erect the entire tower may be used, or short gin poles may be lashed to the part of the tower already erected; the gin poles being moved up as the erection

proceeds. Steel tanks are commonly erected from a movable platform suspended inside the tank. A movable swinging platform for the riveters is also swung outside of the tank.

ERECTION TOOLS.—The tools and appliances used in the erection of structural steel vary so much that it will only be possible to give a brief summary together with data not ordinarily available. Many of the tools and appliances used in the erection of structural steel are of standard contruction and may be purchased direct from dealers, so that a detailed description is not necessary.

Design of Erection Tools.—For the design of hoists, derricks, cranes, crane hooks, and other tools used in bridge erection, see Hess's "Machine Design, Hoists, Derricks, Cranes," published by J. B. Lippincott Company.

Hoists.—Hoisting engines may have the boilers attached or may be detached. A self-contained steam hoisting engine is shown in Fig. 1. Gasoline or electric power may be used to advantage where available. For light hoisting the 4-spool engine is commonly used. Data for the standard hoisting engines used by the American Bridge Company are given in Table I.

Winches and Crabs.—For light hoisting winches or crabs operated by hand power may be used. A crab is attached to the mast or boom, while a winch is self-contained. Views of a crab and of a winch are shown in Fig. 1.

HOISTING ROPE.—Either manila rope or wire rope may be used for hoisting.

Manila Rope.—Only the very best new manila rope should be used for hoisting, as manila rope rapidly deteriorates when used and commercial manila rope varies greatly in strength. The weight, ultimate strengths and safe working loads for manila rope are given in Table II. Working loads with a factor of safety of three should only be used with new rope of the best quality.

TABLE I.

STANDARD HOISTING ENGINES. AMERICAN BRIDGE COMPANY.

	Ordinary Rated H. P.	Lead Line Pull Single Line Average Speed, Lb.	Weight with Boiler, Lb.	Drums.		Spools, Size, In.	Boilers.		Bed.	
				Diam., In.	Length, In.		Diam., In.	Length, In.	Width, Ft-In.	Length, Ft-In.
Double Drum, 4 Spool	20 H. P.	5,000	12,000	14	26	17	42	96	5–0	8–0
Double Drum, 4 Spool	35 H. P.	9,000	15,000	14	27	19	46	108	6–0	10–0
6 Spool	45 H. P.	12,000	22,000	16	30	22	50	108	7–0	11–0
8 Spool	60 H. P.	15,000	30,000	16	34	22	54	108	8–0	12–0

TABLE II.

MANILA ROPE. ULTIMATE STRENGTH, WEIGHT AND WORKING STRESS OF BEST MANILA ROPE.

Diameter, In.	Circumference of Rope, In.	Weight 100 Ft. Rope, Lb.	Ultimate Strength, Lb.	Working Load for Derricks.		Minimum Size of Drum or Sheave, In.
				Used Rope, Factor of 6, Lb.	New Rope, Factor of 3, Lb.	
$\frac{1}{2}$	1.57	7	1,800	300	600	....
$\frac{3}{4}$	2.37	17	4,000	670	1,340	....
$\frac{7}{8}$	2.75	24	5,400	900	1,800	....
1	3.14	28	7,200	1,200	2,400	8
$1\frac{1}{4}$	3.93	46	11,200	1,870	3,740	10
$1\frac{1}{2}$	4.71	64	16,000	2,670	5,340	12
$1\frac{3}{4}$	5.50	84	21,600	3,600	7,200	14
2	6.28	115	28,500	4,750	9,500	16
$2\frac{1}{2}$	7.86	175	45,000	7,500	15,000	....
3	9.42	252	64,200	10,700	21,400	....

Knots in Manila Rope.—In a knot no two parts which lie alongside of each other should move in the same direction in case the rope were to slip. A few of the more common knots are shown in Fig. 2 which has been taken from C. W. Hunt Company's book on "Manila Rope."

1. Bight of a rope.
2. Simple or Overhang Knot.
3. Figure 8 Knot.
4. Double Knot.
5. Boat Knot.
6. Bowline, first step.
7. Bowline, second step.
8. Bowline, completed.
9. Square or Reef Knot.
10. Sheet Bend or Weaver's Knot.
11. Sheet Bend with a toggle.
12. Carrick Bend.
13. "Stevedore" Knot completed.
14. "Stevedore" Knot commenced.
15. Slip Knot.
16. Flemish Loop.
17. Chain Knot with toggle.
18. Half-hitch.
19. Timber-hitch.
20. Clove-hitch.
21. Rolling hitch.
22. Timber-hitch and Half-hitch.
23. Black-wall-hitch.
24. Fisherman's Bend.
25. Round Turn and Half-hitch.
26. Wall Knot commenced.
27. Wall Knot completed.
28. Wall Knot Crown commenced.
29. Wall Knot Crown completed.

"The bowline 7 is one of the most useful knots; it will not slip, and after being strained is easily untied. Commence by making a bight in the rope, then put the end through the bight and under the standing part as shown in Fig. 2, then pass the end again through the bight, and haul tight.

"The square or reef knot 9 must not be mistaken for the 'granny' knot that slips under a strain. Knots 8, 10 and 13 are easily untied after being under strain. The knot 13 is useful when the rope passes through an eye and is held by the knot, as it will not slip, and is easily untied after being strained.

TABLE III.

CRUCIBLE STEEL HOISTING ROPE. WEIGHT, ULTIMATE STRENGTH AND WORKING LOADS OF WIRE ROPE COMPOSED OF 6 STRANDS AND A HEMP CENTER, 19 WIRES TO THE STRAND.

Diameter, In.	Approximate Circumference, In.	Weight per Ft., Lb.	Approximate Breaking Stress, Lb.	Safe Working Stress for Derricks, Factor of 4, Lb.	Minimum Size of Drum or Sheave.	
					Derricks, In.	Rapid Hoisting, In.
3/8	1 1/8	0.22	10,000	2,500	6	12
7/16	1 1/4	0.30	13,600	3,400	7 1/2	15
1/2	1 1/2	0.39	17,600	4,400	9	18
9/16	1 3/4	0.50	22,000	5,500	10	21
5/8	2	0.62	27,200	6,800	12	27
3/4	2 1/4	0.89	38,800	9,700	14	36
7/8	2 3/4	1.20	52,000	13,000	18	42
1	3	1.58	68,000	17,000	20	48
1 1/8	3 1/2	2.00	84,000	21,000	22	54
1 1/4	4	2.45	100,000	25,000	24	60
1 3/8	4 1/4	3.00	124,000	31,000	27	66
1 1/2	4 3/4	3.55	144,000	36,000	30	69

"The timber-hitch, 19, looks as though it would give way, but it will not; the greater the strain the tighter it will hold. The wall knot looks complicated; but is easily made by proceeding as follows: Form a bight with strand *a* and pass the strand *b* around the end of it, and the strand *c* around the end of *b*, and then through the bight of *a*, as shown in the engraving 26. Haul the ends taut, when the appearance is as shown in 27. The end of the strand *a* is now laid

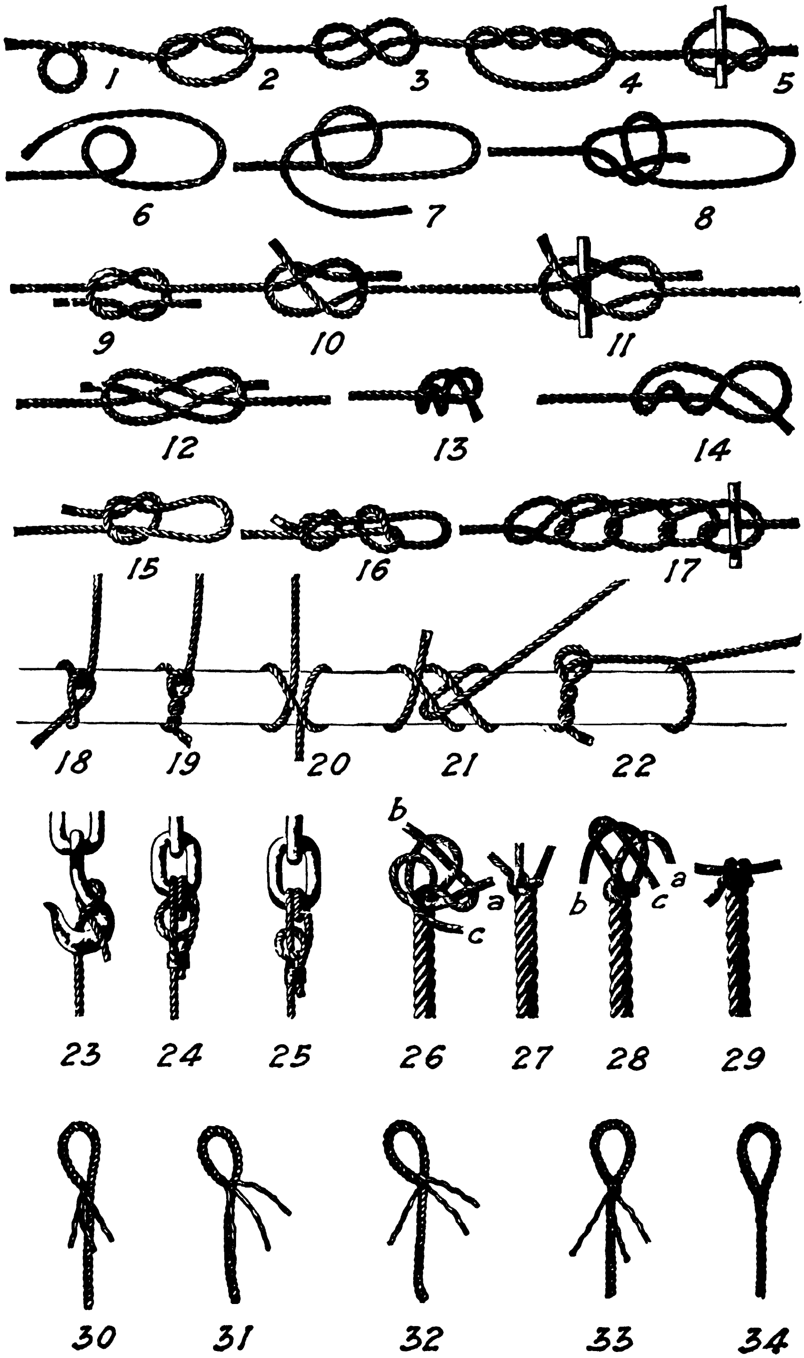


FIG. 2. KNOTS IN MANILA ROPE.

over the centre of the knot, strand *b* laid over *a*, and *c* over *b*, when the end of *c* is passed through the bight of *a*, as shown in 28. Haul all the strands taut, as shown in 29."

The efficiency of a knot will vary from 45 to 75 per cent.

TABLE IV.

PLOUGH STEEL HOISTING ROPE. WEIGHT, ULTIMATE STRENGTH AND WORKING LOADS OF WIRE ROPE COMPOSED OF 6 STRANDS AND A HEMP CENTER, 19 WIRES TO THE STRAND.

Diameter, In.	Approximate Circumference, In.	Weight per Foot, Lb.	Approximate Breaking Stress, Lb.	Safe Working Stress for Derricks, Factor of 4, Lb.	Minimum Size of Drum or Sheave.	
					Derricks, In.	Rapid Hoisting, In.
⅜	1⅛	0.22	11,500	2,870	9	18
7/16	1¼	0.30	16,000	4,000	10½	21
½	1½	0.39	20,000	5,000	12	24
9/16	1¾	0.50	24,600	6,150	14	27
⅝	2	0.62	31,000	7,750	14	33
¾	2¼	0.89	46,000	11,500	16	39
⅞	2¾	1.20	58,000	14,500	18	48
1	3	1.58	76,000	19,000	20	54
1⅛	3½	2.00	94,000	23,500	24	60
1¼	4	2.45	116,000	29,000	28	72
1⅜	4¼	3.00	144,000	36,000	32	81
1½	4¾	3.55	164,000	41,000	36	84

TABLE V.

DATA ON WOODEN BLOCKS FOR MANILA ROPE. AMERICAN BRIDGE COMPANY.

Type of Block.	Nominal Size, In.	Width of Shell, In.	Thickness of Block, In.	Capacity, Tons.	Size of Line, In.	Outside Diameter of Sheave, In.	Weight, Lb.
Single with hook	8	5½	4⅛	2	⅞	4½	15
Double with hook	8	5½	6⅜	4	⅞	4½	20
Single with hook	12	8½	5¼	5	1¼	7½	45
Double with hook	12	8½	8⅜	7	1¼	7½	70
Triple with hook	12	8½	11½	8	1¼	7½	95
Single with hook	14	10¼	6	6	1½	9	70
Double with hook	14	10¼	8⅜	10	1½	9	115
Triple with hook	14	10¼	13¾	12	1½	9	150
Quadruple with shackle . .	14	10¼	16⅛	14	1½	9	190
Single with hook	16	11½	6½	8	1¾	10¼	90
Double with hook	16	11½	10⅛	12	1¾	10¼	140
Triple with hook	16	11½	13¾	15	1¾	10¼	190
Quadruple with shackle . .	16	11½	17⅞	20	1¾	10¼	270
Single with hook	20	14	8¼	15	2 or 2¼	12½	170
Double with hook	20	14	12¾	22	2 or 2¼	12½	230
Triple with hook	20	14	17¼	30	2 or 2¼	12½	360
Quadruple with shackle . .	20	14	21¾	35	2 or 2¼	12½	430
16" snatch block	16	8½	5	5	⅞ or 1¼ or 1½	8	50
20" snatch block	20	9½	6½	8	1½ or 1¾ or 2 or 2¼	9	95

Wire Rope.—Wire hoisting rope is now used for heavy hoisting and in all cases where practicable. Wire rope is much more reliable, gives much greater service, and is much more eco-

nomical and satisfactory than manila rope. Data on crucible cast steel hoisting rope are given in Table III; and data on plough steel hoisting rope are given in Table IV. A factor of safety of 4 should be used for working loads only with derricks or hoists that are not in continuous action. For pile driving and for continuous hoisting a factor of safety of 6 should be used for working loads. Wire ropes used in hoisting are commonly $\frac{5}{8}$, $\frac{3}{4}$ and $\frac{7}{8}$ in. in diameter. The smaller diameters are used for guy lines. For standing guy lines a cheaper wire rope will usually be found satisfactory. Bending stresses in wire ropes are given in Fig. 7, Chapter X.

HOISTING TACKLE.—Blocks for both manila rope and wire rope are made with wooden shells and with steel shells. Blocks up to 12 to 15 tons capacity are commonly provided with hooks; blocks for heavier loads are provided with shackles. Blocks should be well built with adequate bearings and carefully worked out details. The common types of blocks are shown in Fig. 3.

Data on wooden blocks for Manila rope as used by the American Bridge Company are shown in Table V.

Data on steel blocks for wire rope as used by the American Bridge Company are shown in Table VI.

TABLE VI.

DATA ON STEEL BLOCKS FOR WIRE ROPE. AMERICAN BRIDGE COMPANY.

Type of Block.	Width of Shell, In.	Thickness of Block, In.	Capacity, Tons.	Size of Line, In.	Outside Diameter of Sheave, In.	Weight, Lb.
Snatch with hook	17	$7\frac{5}{8}$	8	$\frac{3}{4}$ and $\frac{7}{8}$	14	260
Single with shackle	21	6	10	$\frac{3}{4}$	14	250
Double with shackle	21	$8\frac{3}{4}$	20	$\frac{3}{4}$	14	390
Triple with shackle	21	$11\frac{3}{4}$	30	$\frac{3}{4}$	14	590
Quadruple with shackle	21	$14\frac{1}{2}$	40	$\frac{7}{8}$	14	820
Six sheave with shackle	21	$20\frac{7}{8}$	60	$\frac{7}{8}$	14	1,260

Rigging.—The rigging for lifting loads with wire rope are given in Fig. 4, and for manila rope in Fig. 5. These data are based on experiments made by the American Bridge Company, and have been adopted as standard by the American Bridge Company and the McClintic-Marshall Construction Company.

TABLE VII.

RATIOS OF LOAD TO PULL IN LEAD LINE.

Diam. of Rope, In.	Working Load, Lb.	Manila Rope. Lift per Unit Pull in Lead Line for Tackle with Parts as follows.													
		1	2	3	4	5	6	7	8	9	10	11	12	13	14
$\frac{3}{4}$	1,900	0.86	1.93	2.73	3.48	4.12	4.71	5.23	5.71	6.12	6.50	6.83	7.14	7.40	7.64
$\frac{7}{8}$	2,300	0.83	1.92	2.68	3.37	3.95	4.48	4.92	5.32	5.66	5.96	6.22	6.45	6.64	6.82
1	3,100	0.87	1.93	2.74	3.50	4.16	4.77	5.30	5.80	6.23	6.63	6.98	7.30	7.58	7.85
$1\frac{1}{4}$	4,300	0.83	1.92	2.68	3.37	3.95	4.48	4.92	5.32	5.65	5.96	6.21	6.44	6.63	6.81
$1\frac{1}{2}$	5,900	0.83	1.91	2.67	3.36	3.93	4.45	4.89	5.28	5.61	5.91	6.15	6.38	6.56	6.73
$1\frac{3}{4}$	7,900	0.81	1.91	2.64	3.30	3.84	4.33	4.72	5.08	5.37	5.64	5.85	6.04	6.20	6.34
2	10,300	0.82	1.91	2.65	3.32	3.87	4.37	4.78	5.14	5.45	5.72	5.94	6.15	6.31	6.46
$2\frac{1}{4}$	13,100	0.80	1.90	2.63	3.28	3.80	4.28	4.65	5.00	5.27	5.52	5.72	5.90	6.04	6.17
		Wire Rope.													
$\frac{3}{4}$	16,600	0.86	1.93	2.73	3.47	4.11	4.70	5.20	5.68	6.08	6.46	6.78	7.08	7.34	7.58

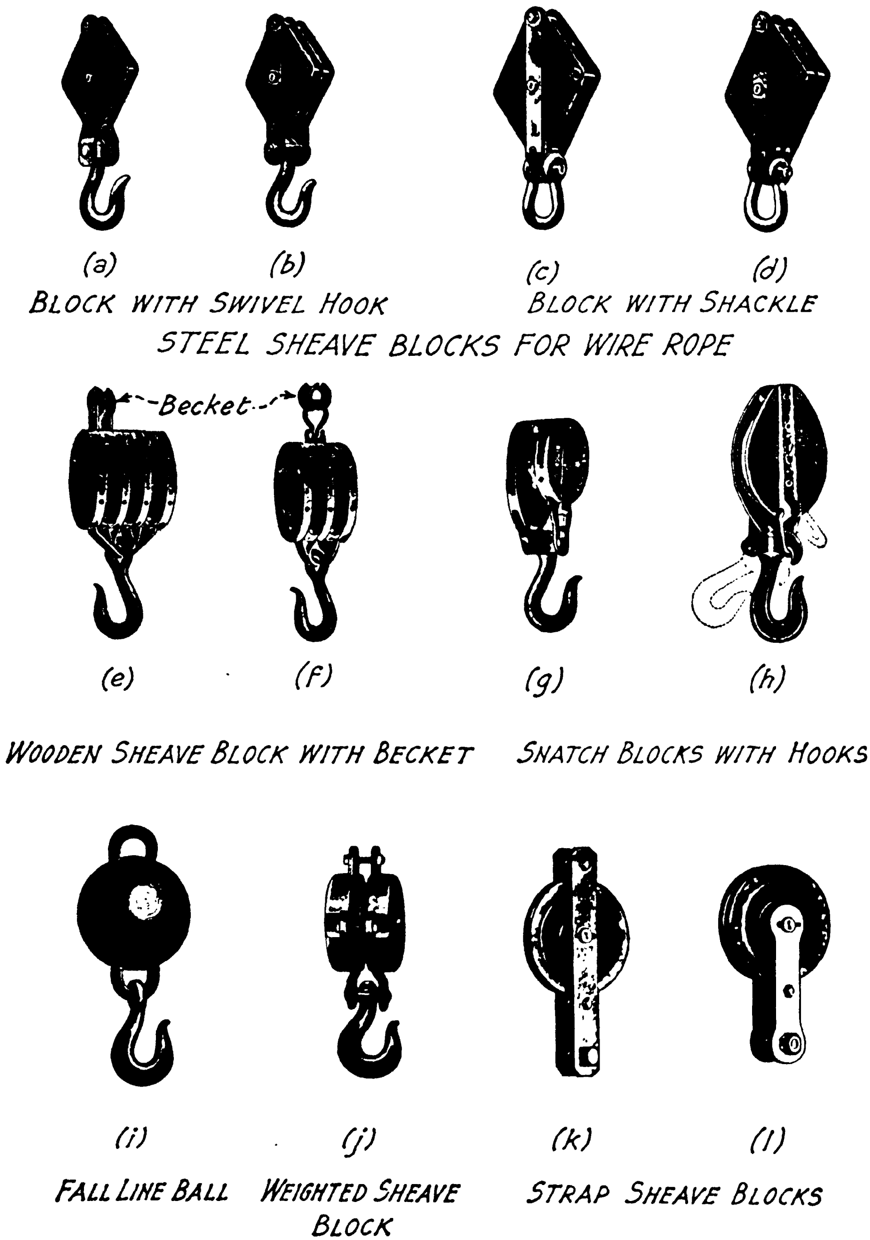


FIG. 3. BLOCKS FOR HOISTING.

Lift Tons	Lead Line Pull-Lbs.	Rigging $\frac{5}{8}''$ Wire Rope	
10	5,700	Double 4 Parts Double	Single 4 Parts Double
20	8,500	Triple 6 Parts Triple	Double 6 Parts Triple
30	10,600	Quadruple 8 Parts Quadruple	Triple 8 Parts Quadruple
40	10,700		6 Sheav' 13 Parts 6 Sheave
60			

Lift Tons	Lead Line Pull-Lbs.	Rigging $\frac{3}{4}''$ Wire Rope	
10	7,400	Double 3 Parts Single	Single 3 Parts Single
20	9,800	Triple 5 Parts Double	Double 5 Parts Double
30	11,700	Quadruple 7 Parts Triple	Triple 7 Parts Triple
40	13,400		Quadrup' 9 Parts Quadruple
60	16,600		6 Sheav' 13 Parts 6 Sheave

Lift Tons	Lead Line Pull-Lbs.	Rigging $\frac{7}{8}''$ Wire Rope	
10	7,500	Double 3 Parts Single	Single 3 Parts Single
20	11,000	Double 4 Parts Double	Single 4 Parts Double
30	13,800	Triple 6 Parts Triple	Double 6 Parts Triple
40	15,000	Quadruple 8 Parts Quadruple	Triple 8 Parts Quadruple
60	19,000		5 Sheav' 11 Parts 5 Sheave

Best Crucible Cast Steel Hoisting Rope: 6 Strand, 19 Wires to a Strand and Hemp Core.

These values are only for tackle as shown. If the lead line is snatched or passes over additional sheaves, capacity diminishes.

LIFTING CAPACITY OF TACKLE
STEEL SHELL BLOCKS
WITH WIRE ROPE

FIG. 4.

Lift Tons	Rigging $1\frac{1}{4}''$ Manila Rope	
4	Single 2 Parts Single	Single 2 Parts
5	Double 3 Parts Single	Single 3 Parts
6	Double 4 Parts Double	Single 4 Parts Double
7	Double 4 Parts Double	Single 4 Parts Double
8	Triple 6 Parts Triple	Double 6 Parts Triple

Lift Tons	Rigging $1\frac{1}{2}''$ Manila Rope	
5	Single 2 Parts Single	Single 2 Parts Single
6	Double 3 Parts Single	Single 3 Parts Single
7	Double 4 Parts Double	Single 4 Parts Double
8	Double 4 Parts Double	Single 4 Parts Double
9	Double 4 Parts Double	Single 4 Parts Double

Lift Tons	Rigging $1\frac{1}{2}''$ Manila Rope	
10	Triple 5 Parts Double	Double 5 Parts Double
11	Triple 6 Parts Triple	Double 6 Parts Triple
12	Triple 6 Parts Triple	Double 6 Parts Triple
13	Quadruple 8 Parts Quadruple	Triple 8 Parts Quadruple
14	Quadruple 8 Parts Quadruple	Triple 8 Parts Quadruple

Lift Tons	Rigging $2''$ Manila Rope.	
20	Triple 6 Parts Triple	Double 6 Parts Triple
22	Triple 6 Parts Triple	Double 6 Parts Triple
24	Quadruple 8 Parts Quadruple	Triple 7 Parts Triple
26	Quadruple 8 Parts Quadruple	Triple 8 Parts Quadruple
28		Quadruple 9 Parts Quadruple

12" Blocks For $1\frac{1}{4}''$ Rope.
Capacity of Blocks
Single with Hook, 5 Tons.
Double with Hook, 7 Tons.
Triple with Hook, 8 Tons.
Approximate pull on lead line, 2 Tons.

14" Blocks For $1\frac{1}{2}''$ Rope.
Capacity of Blocks
Single with Hook, 6 Tons.
Double with Hook, 10 Tons.
Triple with Hook, 12 Tons.
Quadruple with Shackle, 14 Tons.
Approximate pull on lead line, 3 Tons.

20" Blocks For 2" Rope.
Capacity of Blocks
Single with Shackle, 15 Tons.
Double with Shackle, 22 Tons.
Triple with Shackle, 30 Tons.
Quadruple with Shackle, 35 Tons.
Approximate pull on lead line, 5 Tons.

These values are only for tackle as shown. If lead line is snatched or passes over additional sheaves, capacity diminishes.

LIFTING CAPACITY OF TACKLE
WOODEN SHELL BLOCKS WITH MANILA ROPE.

FIG. 5.

Efficiency of Tackle.—The efficiency of rigging as calculated from tests made by the American Bridge Company is given in Table VII. The tables may be used in calculating the loads that can be lifted by tackle as follows:—

Given pull in lead line, to find load lifted—Divide the pull by 1.20 each time line is snatched or passes over sheaves other than those in tackle blocks; multiply quotient by ratio of load to lead line pull, Table VII, and the result is the load lifted. For example, lead line pull of engine = 10,000 lb.; rigging as follows:—2 snatch blocks, 2 sheaves, and 7 parts of $1\frac{1}{2}$ in. line in main falls. Then Load lifted $= \dfrac{10{,}000}{(1.20)^4} \times 4.89 = 23{,}600$ lb. If load to be lifted is given, to find pull in lead line, reverse above operation.

TABLE VIII.

DATA ON CHAINS. AMERICAN BRIDGE COMPANY.

Size, Diam. of Bar, In.	Weight per Foot in Lb.	Outside Lengths of Links in In.	Outside Width of Links in In.	Proof Test in Lb.	Ultimate Strength in Lb.	Working Load in Lb. Factor of 3.	Working Load in Lb. Factor of 4.
$\frac{1}{2}$	2.5	$2\frac{3}{8}$	$1\frac{7}{8}$	7,700	15,000	5,000	3,800
$\frac{5}{8}$	4.10	3	$2\frac{1}{4}$	12,000	23,000	7,600	5,700
$\frac{3}{4}$	6.70	$3\frac{1}{2}$	$2\frac{5}{8}$	17,000	33,000	11,000	8,200
$\frac{7}{8}$	8.37	4	3	22,000	43,000	14,300	10,700
1	10.50	$4\frac{5}{8}$	$3\frac{3}{8}$	29,000	56,000	18,600	14,000
$1\frac{1}{8}$	13.62	$5\frac{1}{8}$	$3\frac{7}{8}$	37,000	71,000	23,600	17,700
$1\frac{1}{4}$	16.00	$5\frac{3}{4}$	$4\frac{1}{4}$	46,000	88,000	29,300	22,000
$1\frac{3}{8}$	19.25	$6\frac{1}{2}$	$4\frac{5}{8}$	55,000	106,000	35,300	26,500
$1\frac{1}{2}$	23.00	7	$5\frac{1}{8}$	66,000	126,000	42,000	31,500
$1\frac{5}{8}$	28.00	$7\frac{1}{4}$	$5\frac{1}{2}$	74,000	141,000	47,000	35,200

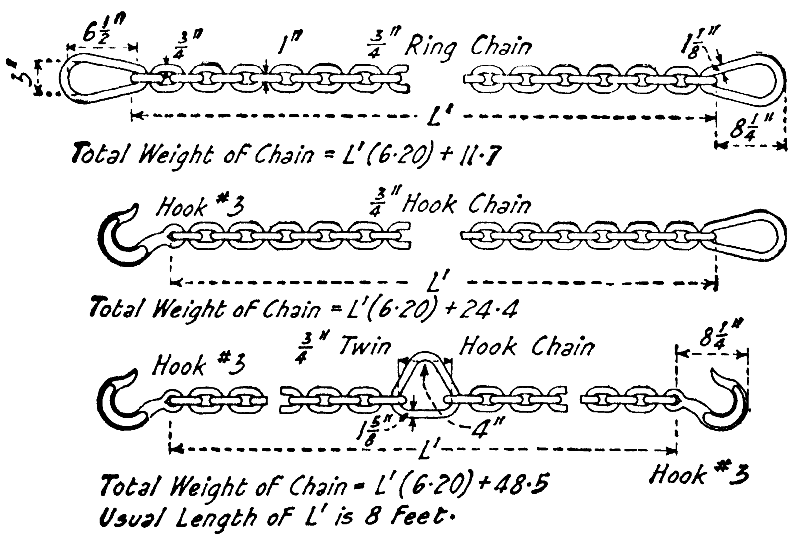


FIG. 6. CHAINS.

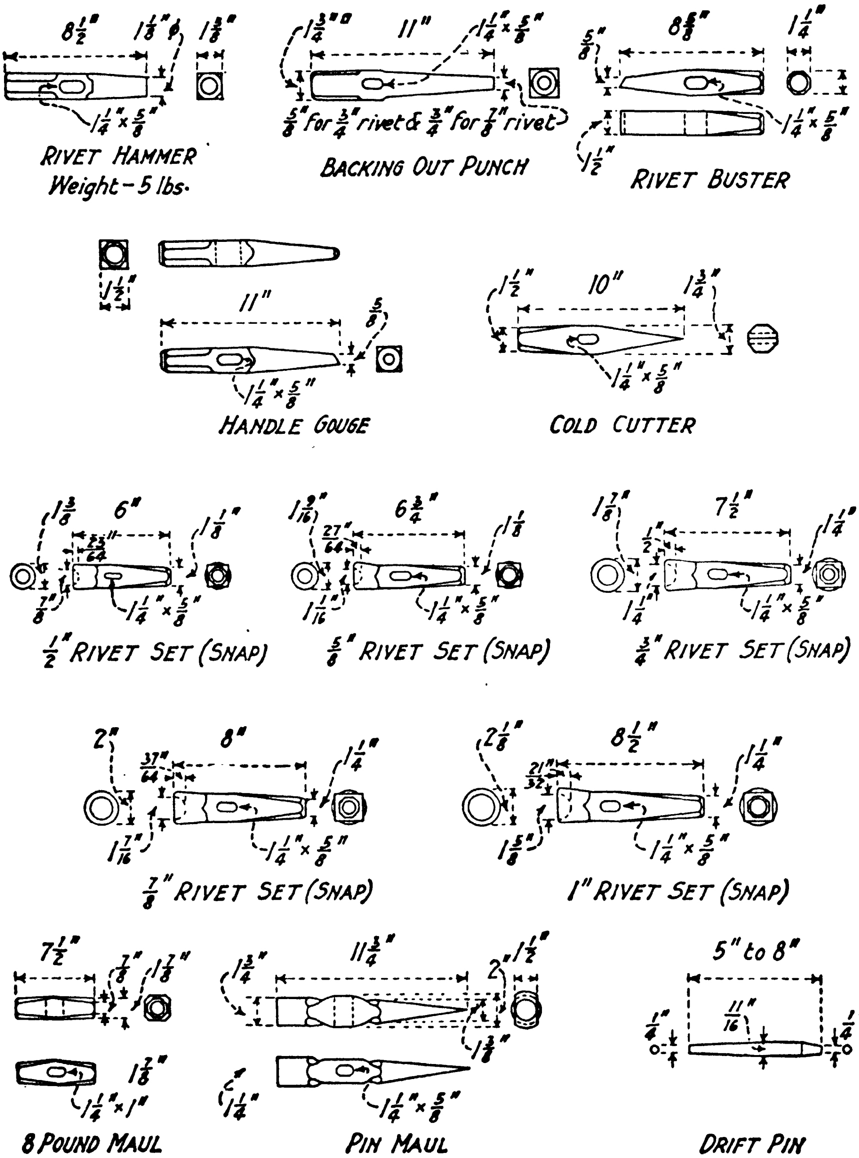


FIG. 7. TOOLS FOR STEEL ERECTION. AMERICAN BRIDGE COMPANY

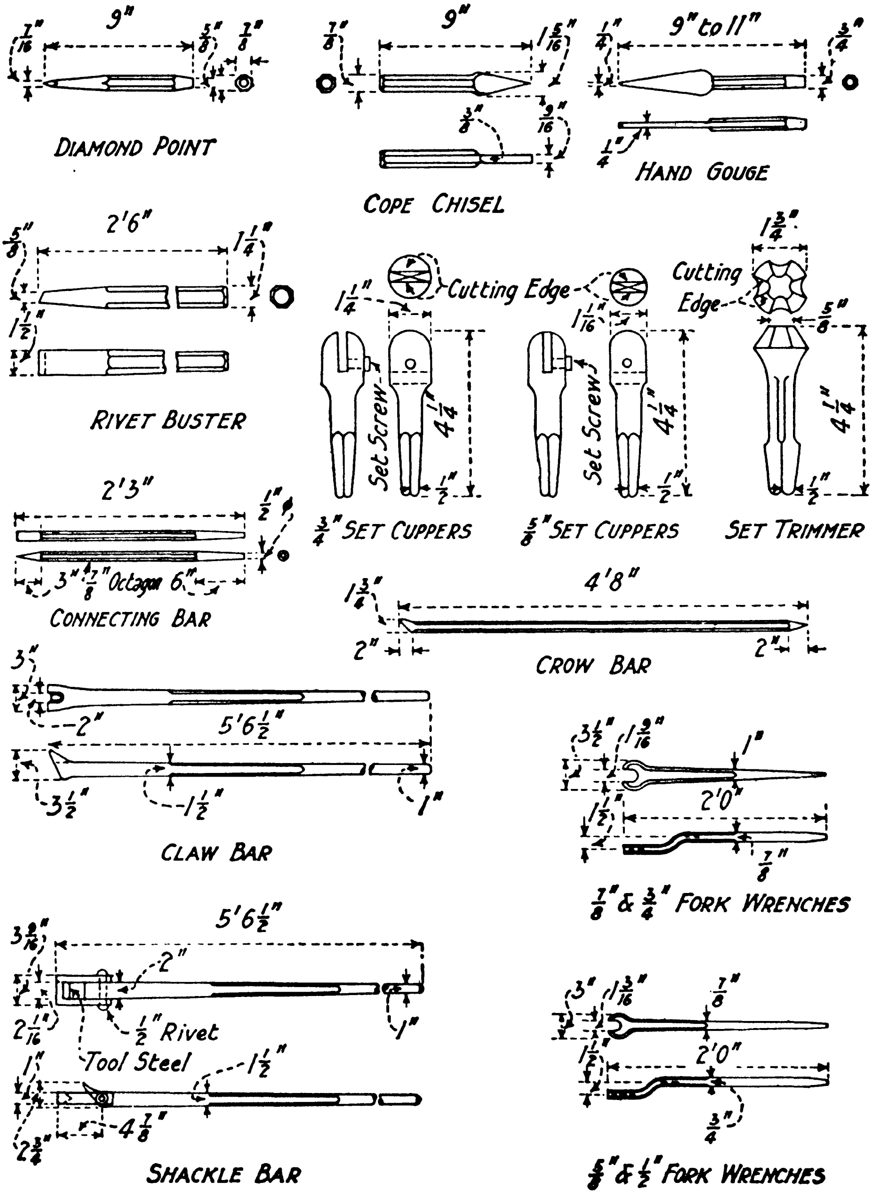


FIG. 8. TOOLS FOR STEEL ERECTION. AMERICAN BRIDGE COMPANY.

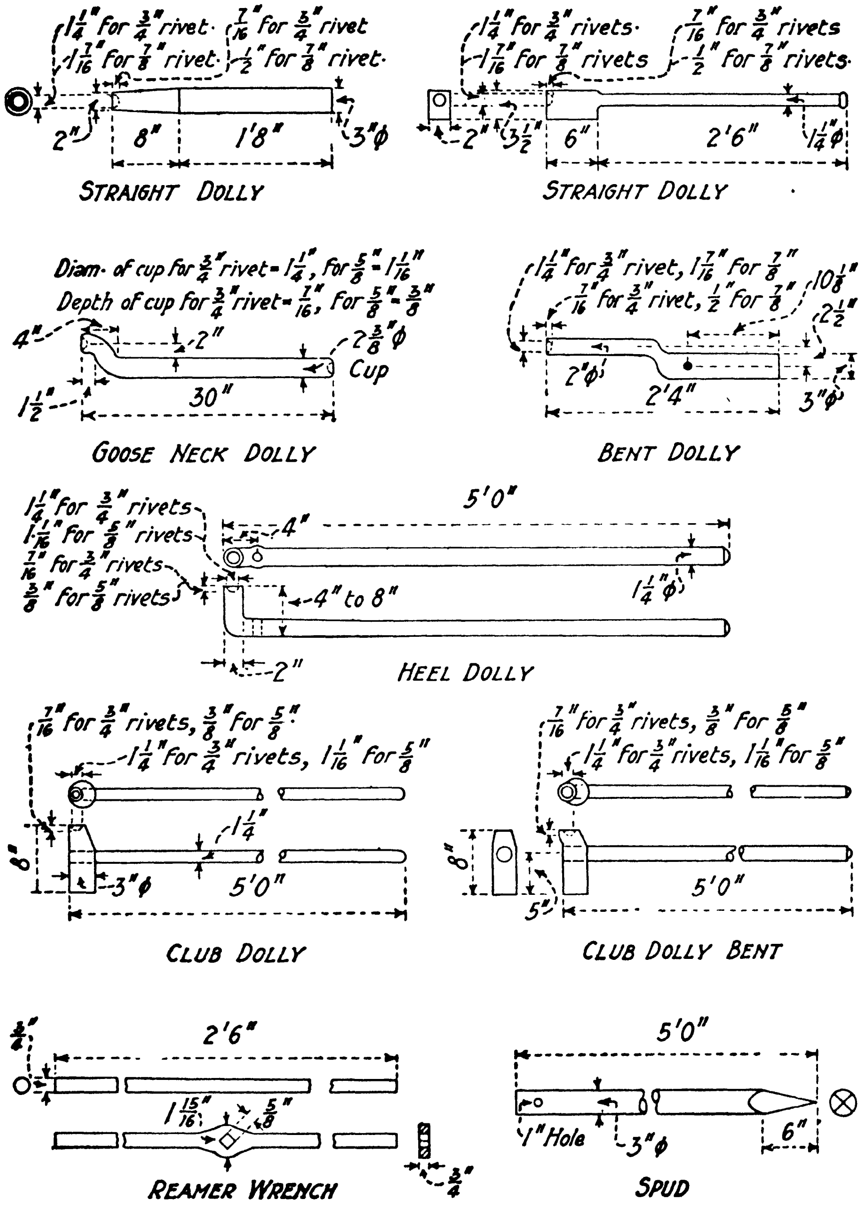


FIG. 9. TOOLS FOR STEEL ERECTION. AMERICAN BRIDGE COMPANY.

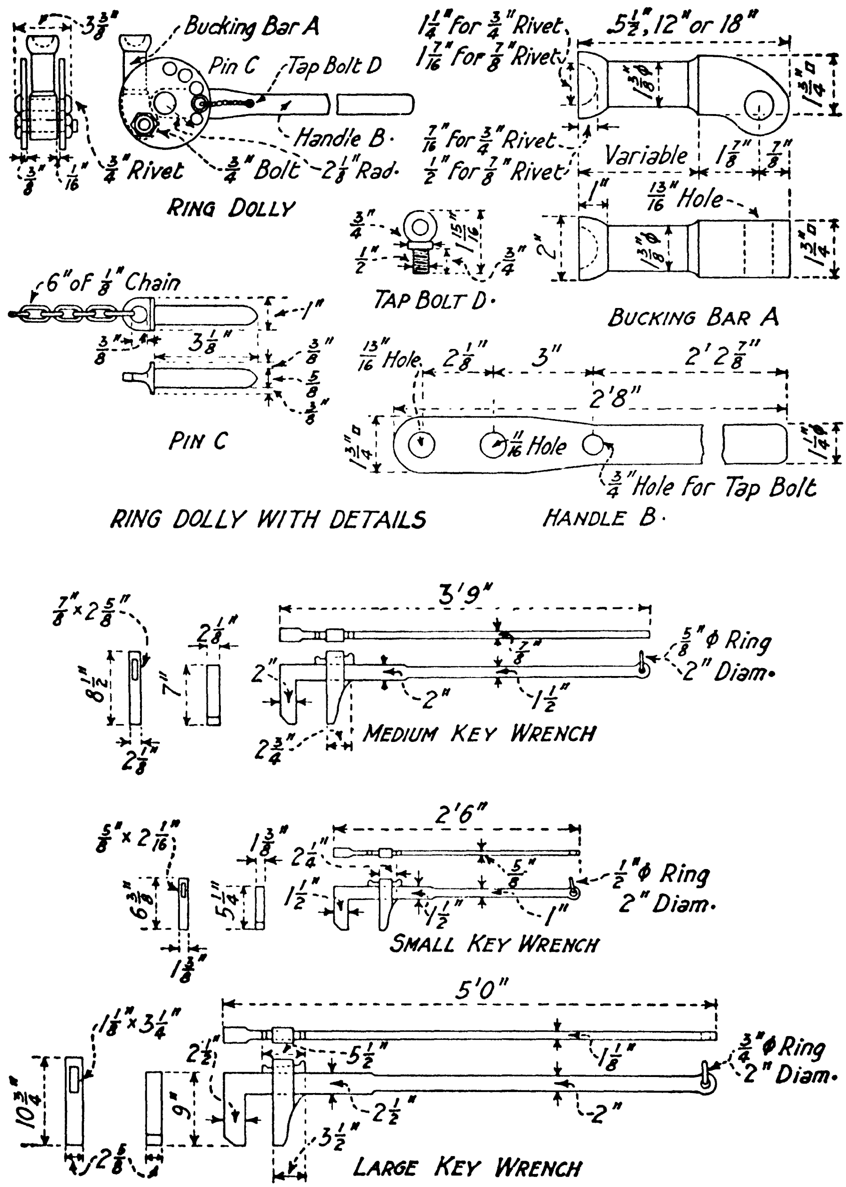


FIG. 10. TOOLS FOR STEEL ERECTION. AMERICAN BRIDGE COMPANY.

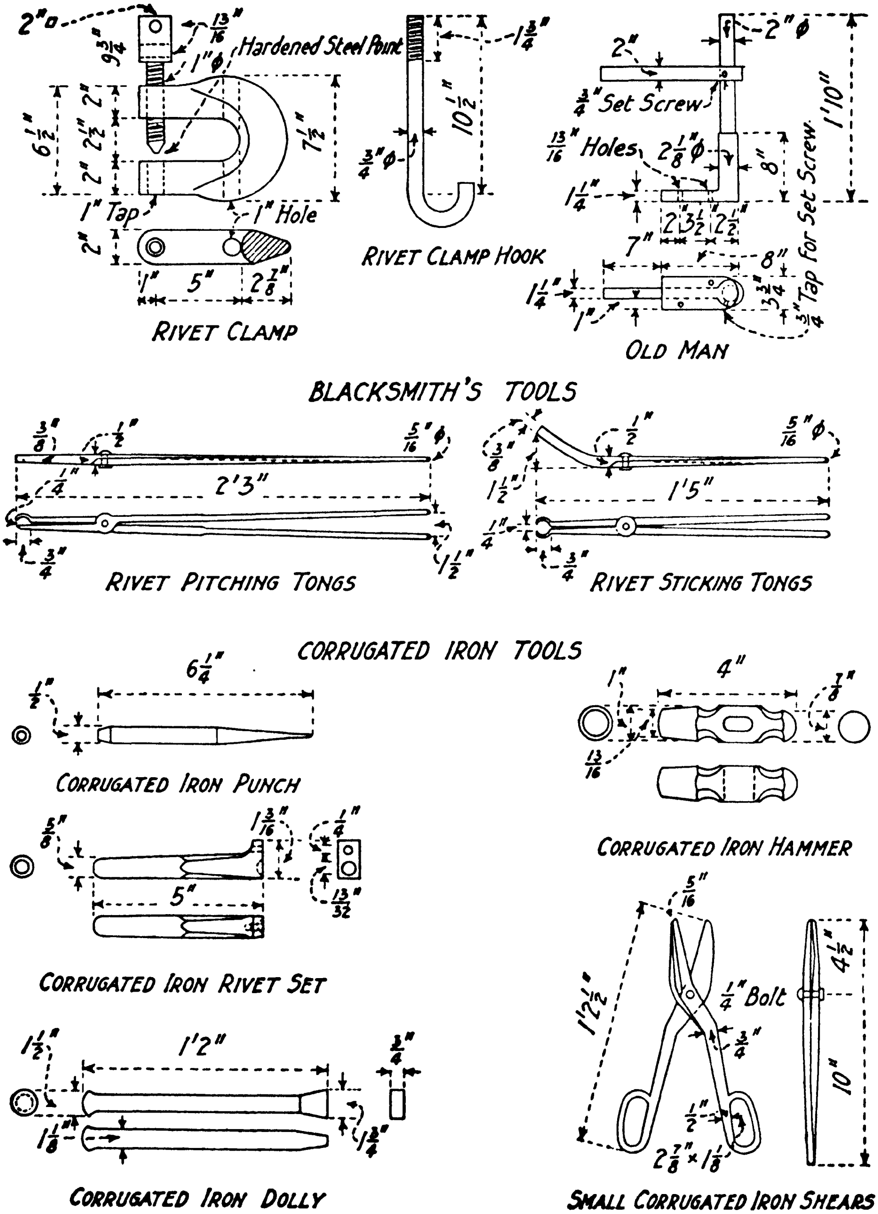


FIG. 11. TOOLS FOR STEEL ERECTION. AMERICAN BRIDGE COMPANY.

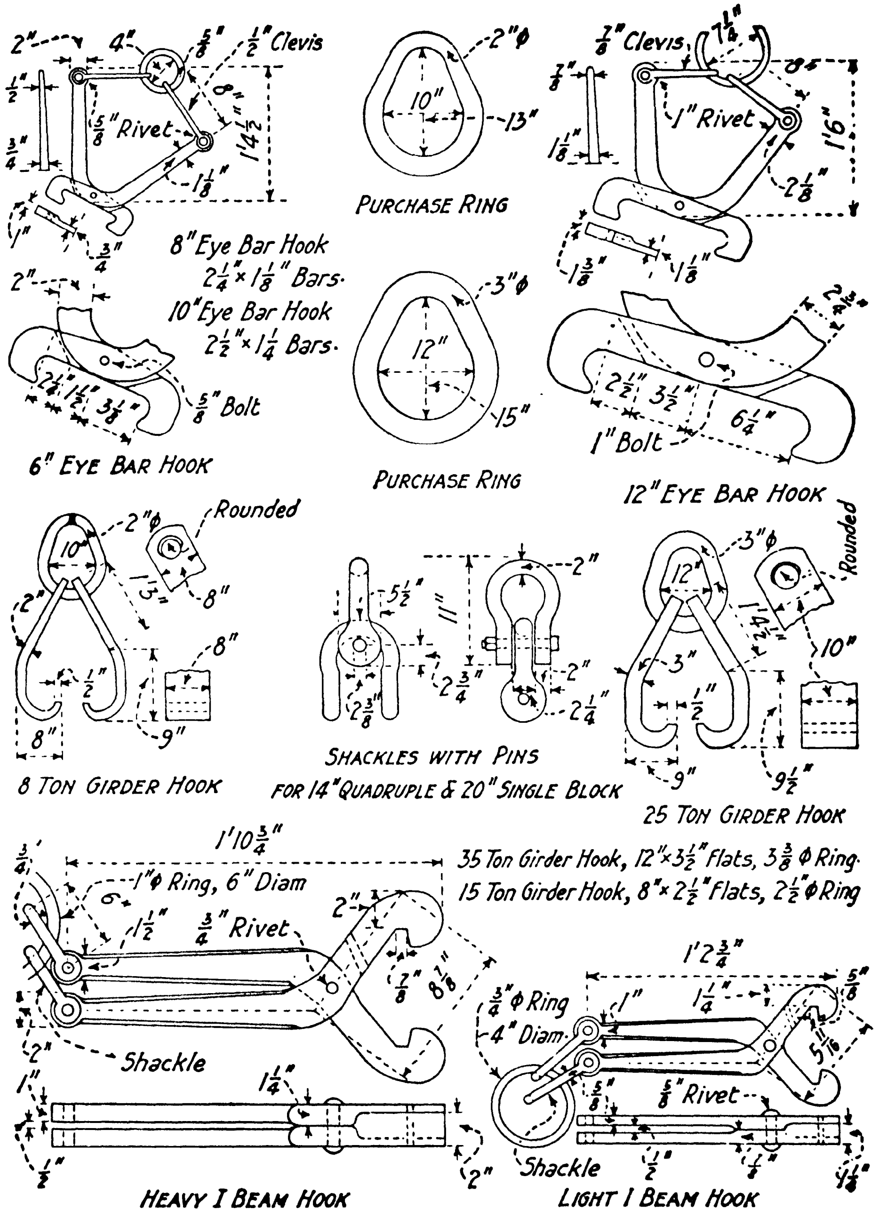


FIG. 12. TOOLS FOR STEEL ERECTION. AMERICAN BRIDGE COMPANY.

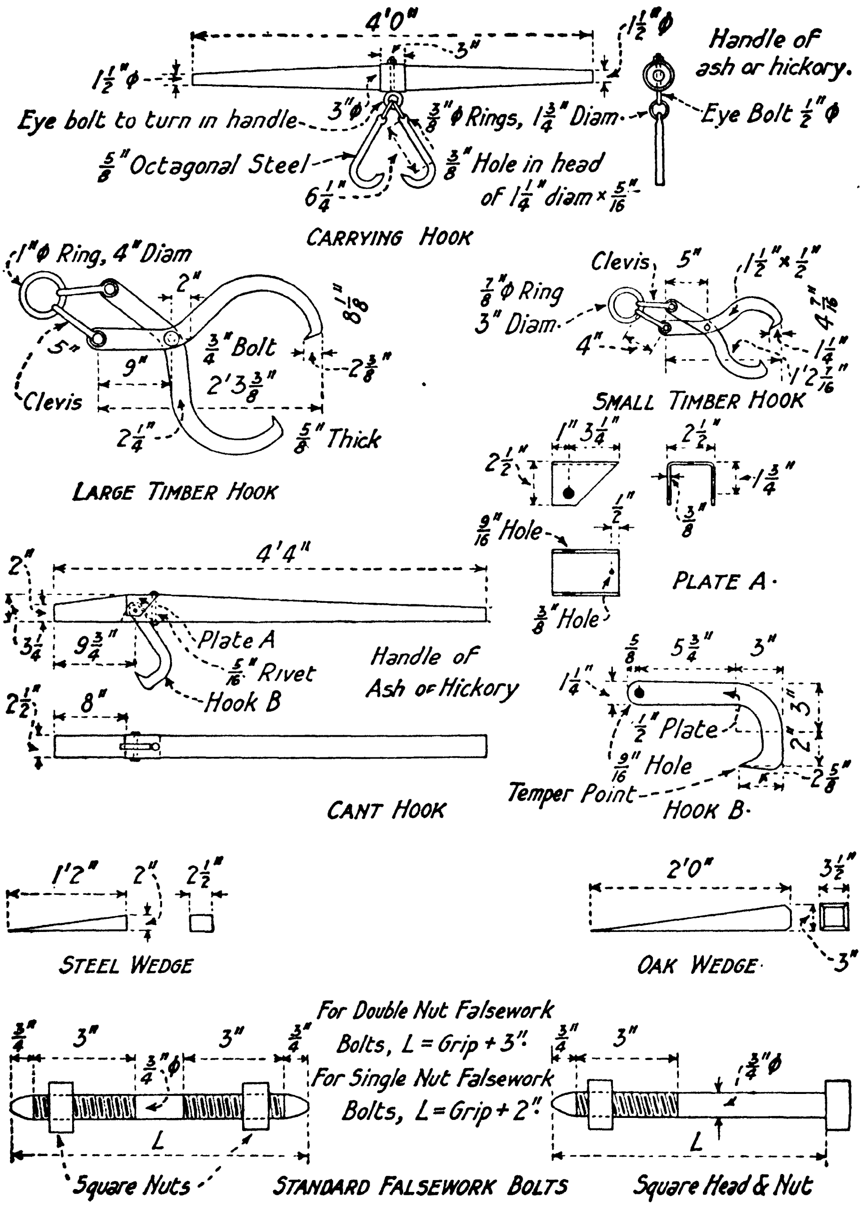


FIG. 13. TOOLS FOR STEEL ERECTION. AMERICAN BRIDGE COMPANY.

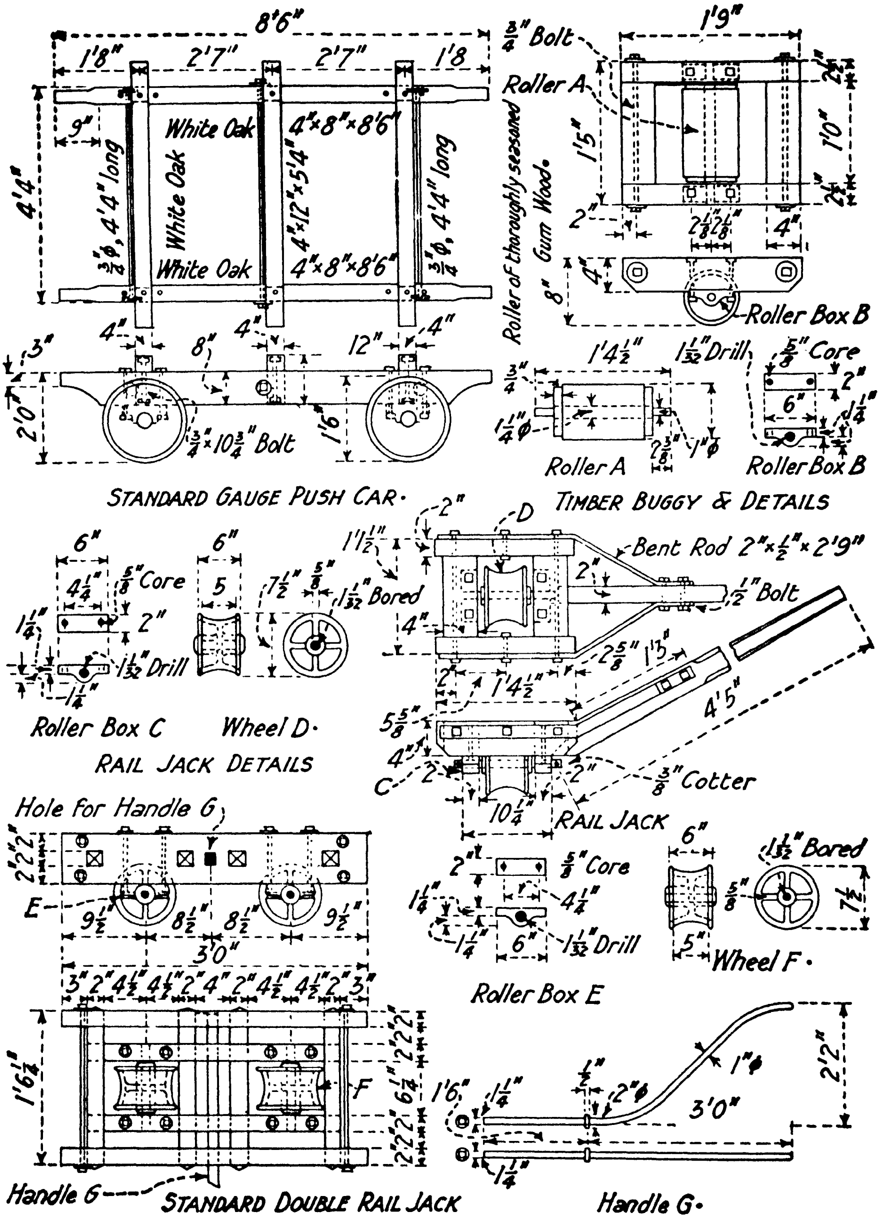


FIG. 14. TOOLS FOR STEEL ERECTION. AMERICAN BRIDGE COMPANY.

Chains.—Chains should be made of the best grade of double refined iron, and should be fabricated with great care. Details of a $\frac{3}{4}$-in. ring chain; a $\frac{3}{4}$-in. hook chain, and of a $\frac{3}{4}$-in. twin hook chain, as made for the American Bridge Company, are given in Fig. 6, and data on chains are given in Table VIII.

Jacks.—Hydraulic and power lifting jacks of the necessary capacity should be provided.

Miscellaneous Tools.—In addition to the standard tools required by bridge carpenters and by the blacksmiths many special tools are required by structural steel erectors. The most important special tools required in steel erection as used by the American Bridge Company are

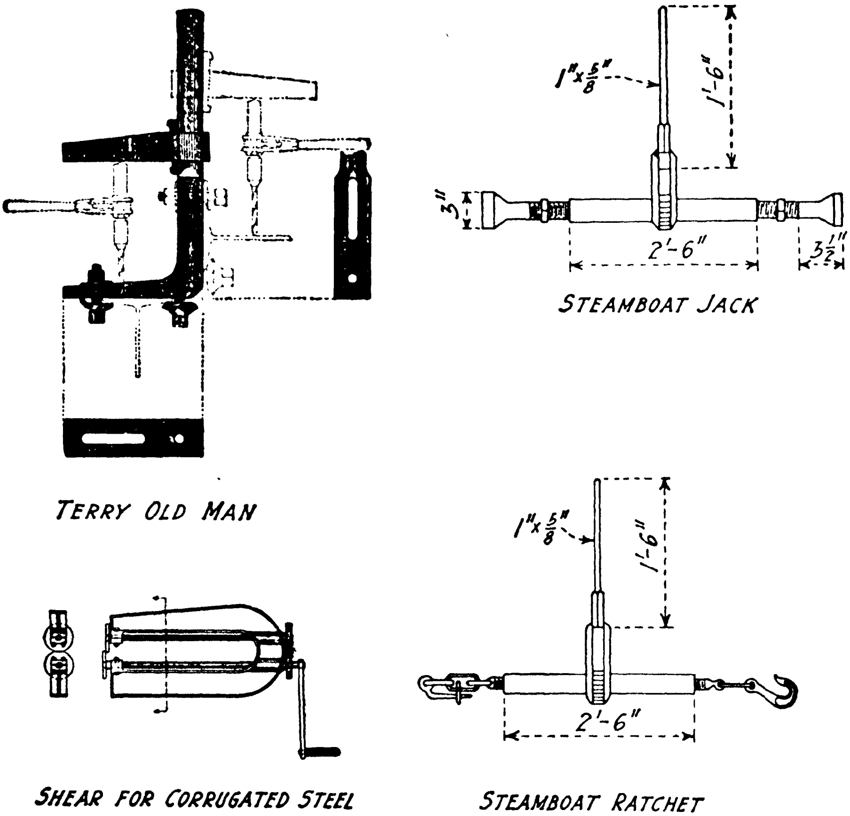


FIG. 15. MISCELLANEOUS TOOLS FOR STEEL ERECTION.

given in Fig. 7 to Fig. 14. An improved "old man" as used by Terry and Tench is shown in Fig. 15. A corrugated rolling shear, and a steamboat jack and a steamboat ratchet are also shown in Fig. 15. The special tools used by the Chicago Bridge and Iron Company for the erection of elevated tanks are given in Fig. 16 and Fig. 17.

LIST OF TOOLS.—The tools required for any job will depend upon the size of the work, the number of men employed, and upon local conditions. A complete list of the tools that are commonly used by structural steel erectors is given in Table IX

Actual lists of the tools used for the erection of a steel railway bridge, a steel highway bridge, and a steel mill building are given in Table X, Table XI, and Table XII, respectively.

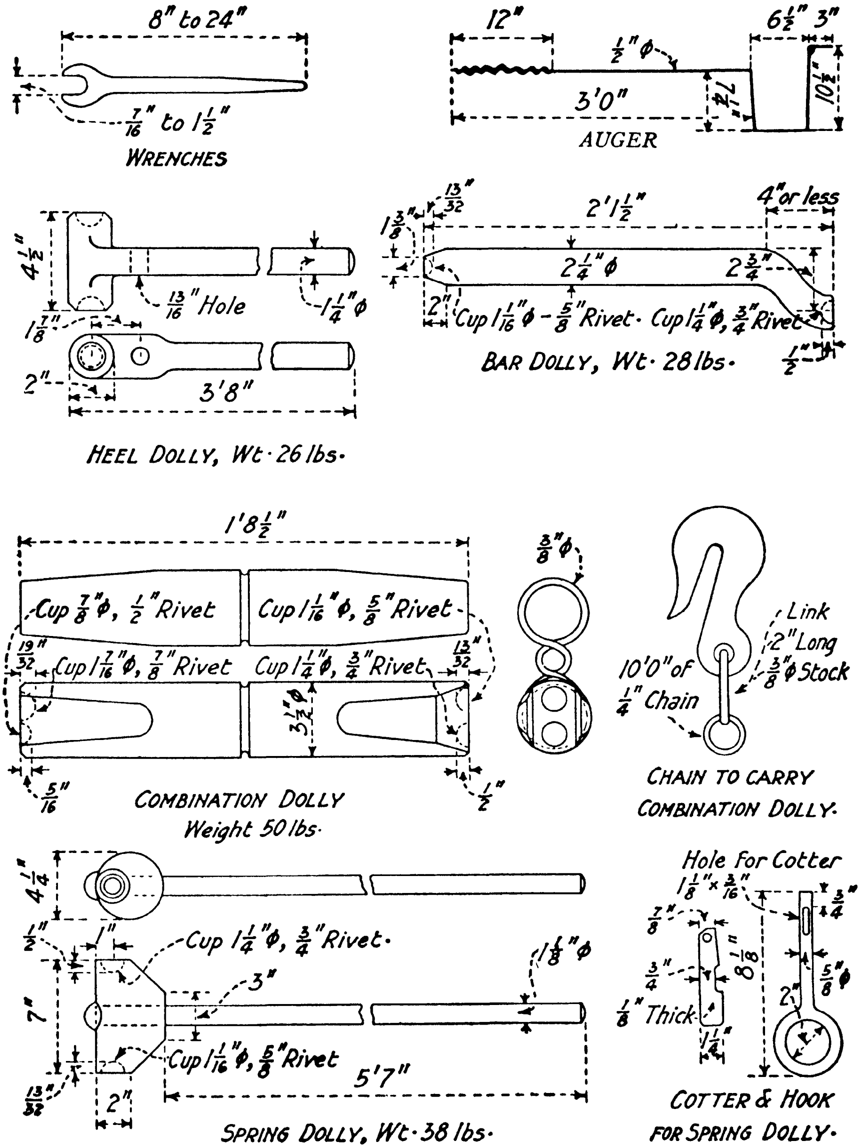
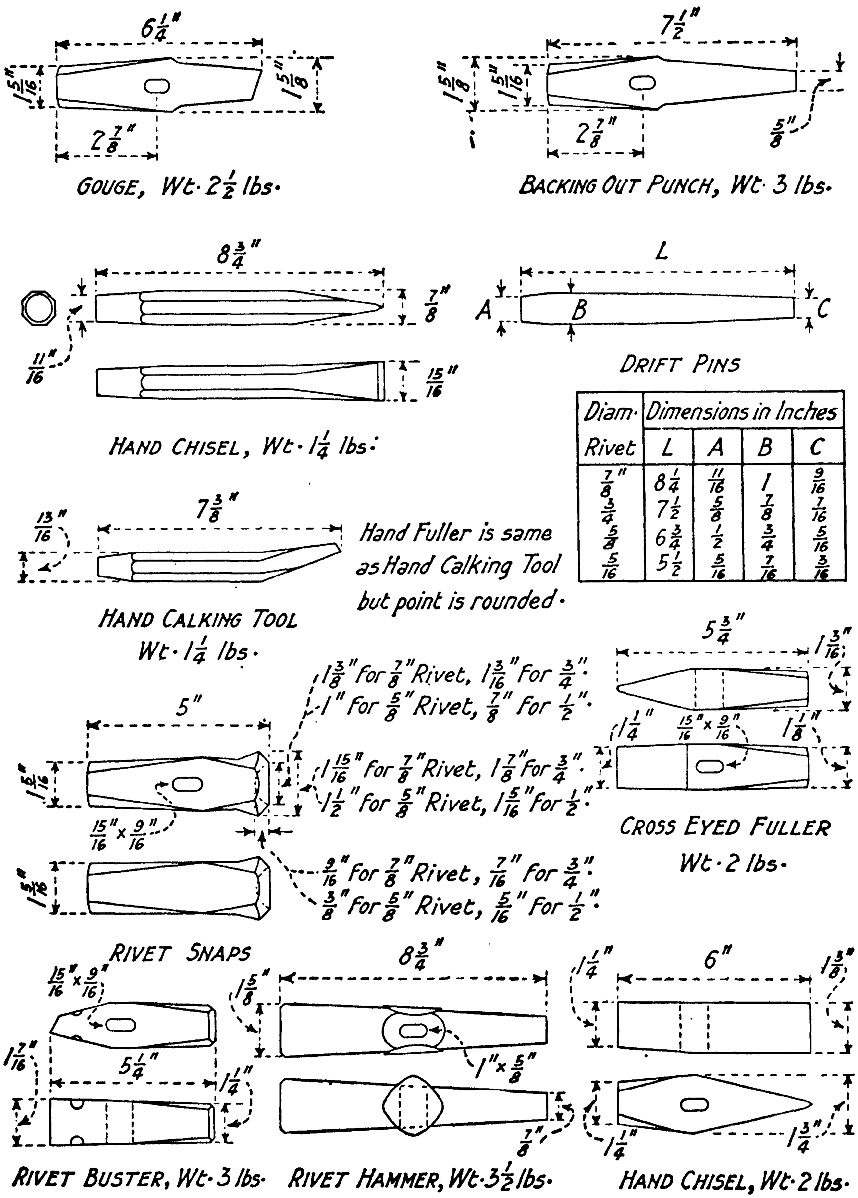


FIG. 16. TOOLS FOR ERECTION OF ELEVATED TANKS. CHICAGO BRIDGE & IRON COMPANY.



Diam. Rivet	Dimensions in Inches L	A	B	C
$\frac{7}{8}$"	$8\frac{1}{4}$	$\frac{11}{16}$	1	$\frac{9}{16}$
$\frac{3}{4}$	$7\frac{1}{2}$	$\frac{5}{8}$	$\frac{7}{8}$	$\frac{7}{16}$
$\frac{5}{8}$	$6\frac{3}{4}$	$\frac{1}{2}$	$\frac{3}{4}$	$\frac{5}{16}$
$\frac{5}{16}$	$5\frac{1}{2}$	$\frac{5}{16}$	$\frac{7}{16}$	$\frac{3}{16}$

FIG. 17. TOOLS FOR ERECTION OF ELEVATED TANKS. CHICAGO BRIDGE & IRON COMPANY.

TABLE IX.

List of Erection Tools for Structural Steel.
American Bridge Company.

Name.
Adzes.
Air Chippers.
Air Compressors
Air Drills.
Air Pumps.
Air Reamers.
Air Receivers.
Anchors.
Angle Bars for R. R. Rails.
Anvils.
Auger Bits.
Augers (ship) $\frac{11}{16}$ in. to $1\frac{1}{16}$ in.
Axes.
Axes (Hand).
Backing Out Punches.
Balance Beams.
Bars, Chisel.
Bars, Claw.
Bars, Connecting.
Bars, Crow.
Bars, Pinch.
Bellows.
Bits for Braces.
Blacksmith Blowers.
Blacksmith Hand Tools.
Blocks (8, 10, 12, 14, 16, 18) in. Single.
Blocks (8, 10, 12, 14, 16, 18) in. Double.
Blocks (14, 16, 18, 20) in., 3 Sheave.
Blocks, 4 Sheave.
Blocks (8, 10, 12, 14, 16, 18, 20) in. (Snatch) Gate.
Blocks (1, 2, 3, 4, 6) Sheave, Wire Rope.
Boats (give kind).
Boilers (only).
Boring Machines.
Braces (Carpenter).
Branding Irons.
Brushes (Paint).
Brushes (Wire).
Buckets.
Car Axles.
Cars, Camp.
Cars, Derrick.
Cars, Flat.
Cars, Lever.
Cars, Push.
Cars, Tool.
Car Wheels.
Center Punches.
Chains, ($\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$, $\frac{7}{8}$) in. Hook & Ring, — ft. long.
Chains, 1 in. Hook & Ring, — ft. long.
Chains, $\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$, 1 in., two rings, — ft. long.
Chisels, Cope.
Chisels, Framing.
Clevises.
Cold Chisels.
Currugated Iron Cutters.
Corrugated Iron Dolly Bars.
“ “ Hammers.
“ “ Punches.
Corrugated Iron Rivet Sets.
“ “ Shears.
Crabs, Single Gear Iron Frame A—Flat.
Crabs, Double Gear Iron Frame A—Flat.
Crabs, Single Gear Wooden Frame A—Flat.
Crabs, Double Gear Wooden Frame A—Flat.
Cutters, Handle.
Derricks.
Derrick Balls Overhauling.
“ Booms (Steel).
“ Booms (Wood).
“ Boom Bands, 2 Links.
“ “ Foot Blocks.
“ “ & Mast Angles.
“ “ Bearing Plates.
“ “ Pins.
“ “ Plates.
“ Foot Blocks.
“ Goose Necks.
“ Gudgeon Pins.
“ Masts (Steel).
“ Masts (Wood).
“ Mast Band.
“ Mast Band, one link.
“ Mast Seat.
“ Round Spiders.
“ Long Spiders, Two Guys.
“ “ “ One Guy.
Diamond Points.
Dolly Bars, Bent.
“ “ Club.
“ “ Goose Necks.
“ “ Heel.
“ “ Spring.
“ “ Straight.
Drawing Knife.
Drilling Machine (Portable).
Drift Pins ($\frac{9}{16}$, $\frac{11}{16}$, $\frac{13}{16}$, $\frac{15}{16}$) in. diameter.
Drills, Flat.
Drills (Stone).
Drills (Twist).
Engine and Boiler.
Eye Bolts.
Files.
Forges (not rivet).
Gauges (Track).
Gin poles (Wood) Gas Pipe, Shoes.
Grind Stone.
Guy Clamps.
Guy Rods.
Guy Wire.
Hammers (Chipping).
Hand Gouges.
Handle Gouges.
Handles—Hammer, Maul, Axe, Adze, Pick.
Hatchets.
Hook for I Beams—Large, Medium, Small.
Hooks, Cant.
Hooks for Eye-Bars.
Hooks, Girder.

TABLE IX.—*Continued.*

Name.	Name.
Hooks for Heavy Chord.	Reamers—$\frac{11}{16}$, $\frac{13}{16}$, $\frac{15}{16}$, $1\frac{1}{16}$ in.
Hooks for holding on.	Reamer Handles.
Hooks, Scaffold.	Rivet Busters.
" Stringer.	" Clamps.
" Timber.	" Clamp Hooks.
Horse Powers.	" Forges.
Hose, Air Drill.	" Gouges.
" Rubber.	" Hammers.
" Steam.	" Sets for—$\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$, $\frac{7}{8}$, 1, in. Rivets (Hand).
" Bands.	" Sets for—$\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$, $\frac{7}{8}$, 1, in. Rivets (Pneumatic).
" Couplings.	
Jacks, Hydr.—Capacity.	Set Cuppers.
" Norton.	Set Gouges, Standard.
" Rail, Double.	Set Rivet Tongs.
" Rail, Single.	Set Trimmers.
" Steamboat.	Spikes.
" Steamboat Pull.	Rollers.
" Steamboat Pushing.	Roofing Sets.
" Screw.	Rope, Manila—$\frac{3}{4}$, 1, $1\frac{1}{4}$, $1\frac{1}{2}$, 2 in.
" Track.	Rope Lashing, Manila.
Kettles, Iron.	Rope Slings, Manila.
Ladles.	Rope, Wire Hoisting.
Lag Screws.	Saws, Crosscut.
Ladders.	Saws, Hand.
Lanterns.	Saw Frames, Hack.
Levels (Spirit).	Saws, One Man.
Locks.	Saw Sets (Crosscut).
Marking Pot.	Screw Drivers.
Mattocks.	Shackles.
Mauls, Spike.	Sheaves,—in. dia.
Mauls, Steel (8, 9, 12, 16, 18; 20) lb.	Shovels.
Nails.	Squares (Carpenter).
Oars.	Stock and Dies.
Oar Locks.	Stoves.
Oil Cans.	Sulphur Pot.
Old Man.	Tape Lines.
Picks.	Tarpaulins.
Pike Poles.	Timber Buggies.
Pile Hammers.	Tool Boxes.
" Driver Leads.	" Steel, Octagon.
" Rings.	" Steel, Round.
" Ring Hooks.	" Steel, Square.
Pins, Cotter.	Traveler Corner Irons.
Pipe Cutters.	" Plates.
Pipe, Iron.	" Rods.
Pipe Tongs.	" Wheels, Standard.
Planes.	Traveler Wheels.
Plumb Bobs.	" Wheel Boxes.
Pneumatic Bucker-up.	Travelers (Wood).
Pneumatic Hammer.	Travelers (Steel).
Pump, Boat, Galvanized Iron.	Turnbuckle Rods.
Pump, Centrifugal.	Tuyere Irons.
" Force.	Valves.
" Steam.	Vises.
Punch, Hydraulic.	Wagons.
Punch, Screw.	Wrenches, Chain.
Purchase Rings.	Wrenches, Fork—$\frac{1}{2}$, $\frac{3}{8}$, $\frac{3}{16}$, $\frac{7}{8}$, in.
Rails (Steel).	Wrenches, Key—large, medium, small.
Rail Splice Plates.	Wrenches, Monkey.
Rail Buggies.	Wrenches, S.
Rams.	Wrenches, Stillson.
Ratchets.	Wedges.

TABLE X.

LIST OF TOOLS FOR ERECTION OF STEEL RAILROAD BRIDGE CONSISTING OF SEVERAL 75-FT. PLATE GIRDERS, A 180-FT. THROUGH SPAN, AND AN 80-FT. VERTICAL LIFT SPAN, INTERNATIONAL FALLS, MINNESOTA. MINNEAPOLIS STEEL & MACHINERY CO.

Quantity.	Name and Size of Tool.	Quantity.	Name and Size of Tool.
3	Augers, Ship, $\frac{13}{16}$ in.	3	Forges, Complete.
2	Adz.	3	Files.
1	Axe, Hand.	2	Gouges, Hand.
2	Anvils.	3	Gouges, Handle.
3	Bars, Crow.	3	Hack Saws and Blades.
1	Bars, Claw.	1	Hammer, 7 lb.
2	Bits, $\frac{5}{8}$ in.	1	Hammer, Claw.
1	Box, Tool.	2	Hammers, Blacksmith, 5 lb.
2	Braces.	16	Handles.
1	Brushes, Wire.	7	Hooks, Scaffold.
7	Brushes, Paint.	1	Hose, Air, $\frac{3}{4}$ in., 700 ft.
1	Block, Steel, Snatch, 10 in.	9	Hose, Water, $\frac{1}{2}$ in. $\times$ 50 ft.
3	Block, Steel, Snatch, 12 in.	4	Jack, Screw, $2\frac{1}{2}$ in. $\times$ 16 in.
3	Block, Steel, Snatch, Wire Rope, 12 in.	1	Jack, Track.
1	Block, Steel, Single, Wire Rope, 12 in.	2	Jack, Stone.
2	Block, Steel, Single, Wire Rope, 14 in.	1	Jack, Hydraulic, 15 ton.
2	Block, Steel, 4 Part, Wire Rope, 16 in.	2	Lanterns.
4	Block, Steel, Double, Wire Rope, 18 in.	1	Level.
4	Block, Steel, Double, Wire Rope, 12 in.	1	Man, Old.
2	Block, Steel, Triple, Wire Rope, 12 in.	4	Punches, Backing Out.
4	Block, Wood, Snatch, 10 in.	3	Punches, Screw (Frame).
2	Block, Wood, Snatch, 12 in.	1	Pipe Vise.
1	Block, Wood, Single, Tackle, 8 in.	1	Pick.
1	Block, Wood, Single, Tackle, 10 in.	12	Drift Pins, $\frac{5}{8}$ in.
1	Block, Wood, Single, Tackle, 12 in.	10	Drift Pins, $\frac{3}{4}$ in.
6	Block, Wood, Double, Tackle, 8 in.	4	Drift Pins, $\frac{7}{8}$ in.
4	Block, Wood, Double, Tackle, 10 in.	1	Pail, Water.
2	Block, Wood, Double, Tackle, 12 in.	2	Ratchets.
1	Block, Wood, Triple, Tackle, 12 in.	1	Receiver, Air, 30 in. $\times$ 60 in.
3	Block, Wood, Triple, Tackle, 14 in.	1,400 ft.	Rope, Manila, 1 in., 7 pieces.
1	Block, Chain, 5 Ton.	1,300 ft.	Rope, Manila, $1\frac{1}{4}$ in., 5 pieces.
1,200 ft.	Cable, Wire, $\frac{1}{2}$ in.	420 ft.	Rope, Manila, 2 in., 1 piece.
300 ft.	Cable, Wire, $\frac{5}{8}$ in.	640 ft.	Rope, Manila, 2 in., 1 piece.
100 ft.	Cable, Wire, $\frac{7}{8}$ in., galvanized.	275 ft.	Rope, Manila, 2 in., 1 piece.
2	Chains, $\frac{5}{8}$ in., 23 ft. long.	565 ft.	Rope, Manila, 1 in., 2 pieces.
1	Chains, $\frac{3}{4}$ in., 14 ft. long.	4	Rope, Manila, Lashings.
2	Chains, $\frac{5}{8}$ in., 12 ft. long.	1	Stock and Dies, Blacksmith.
2	Chains, $\frac{1}{2}$ in , 12 ft. long.	1	Stock and Dies, Pipe.
12	Clamps, Cable, $\frac{1}{2}$ in.	6	Snaps, Rivet, $\frac{5}{8}$ in.
10	Clamps, Cable, $\frac{7}{8}$ in.	6	Snaps, Rivet, $\frac{3}{4}$ in.
8	Clamps, Cable, $\frac{5}{8}$ in.	4	Snaps, Rivet, $\frac{7}{8}$ in.
4	Clamps, Rivet.	3	Saws, Cross Cut.
2	Chisels, Round Nose.	2	Saws, Hand.
1	Chisels, Cold.	1	Shovels, No. 2.
5	Cutters.	4	Shovels, Snow.
3	Cant Hooks.	1	Square.
1	Compressor, Air.	13	Shackles
1	Derrick, 12 ton.	2	Trucks, Dolly.
1	Dolly, Timber.	3	Tongs, Blacksmith.
1	Dolly, Goose Neck.	4	Tongs, Heater
1	Dolly, Straight.	7	Wrenches, Bridge $\frac{3}{4}$ in.
3	Dolly, Spring.	6	Wrenches, Bridge $\frac{7}{8}$ in.
1	Dolly, Wedge.	2	Wrenches, Monkey
1	Dolly, Heel.	1	Heavy Traveler, 12 ton .
5	Drills, Twist, $\frac{15}{16}$ in.	4	Rollers, 10 in. and 12 in.
6	Drills, Twist, $\frac{13}{16}$ in.	5	Pneumatic riveting guns.
6	Drills, Twist, $\frac{11}{16}$ in.	2	28 in Turnbuckles.
1	Drills, $1\frac{1}{2}$ in. $\times$ 4 ft.	2	Stoves.
2	Engine, Hoisting.	27	$\frac{7}{8}$ in. $\times$ 8 in. Step bolts.

TABLE XI.

LIST OF TOOLS FOR THE ERECTION OF 80-FT. SPAN HIGHWAY BRIDGE.
MINNEAPOLIS STEEL & MACHINERY CO.

Quantity.	Name and Size of Tool.	Quantity	Name and Size of Tool.
2	Axes.	1	Man, Old.
2	Axes, Hand.	4	Punches, Backing out.
3	Bits, 1 in., $\frac{3}{4}$ in., $\frac{1}{2}$ in.	1	Pick.
1	Buster.	1	Pump.
1	Box, Tool.	4	Pins, Drift, $\frac{3}{4}$ in.
1	Brace.	6	Pins, Drift, $\frac{5}{8}$ in.
1	Brush, Paint.	2	Pails, Water.
2	Blocks, 10 in.	2	Pile Driver Leads.
1	Block, Single Tackle, 8 in.	1	Pile Driver Hammer.
1	Block, Single Tackle, 10 in.	1	Pile Driver Head Block.
4	Blocks, Double Tackle, 8 in.	1	Pile Driver Nipper
1	Chain, $\frac{3}{8}$ in., 8 ft. long.	1	Ratchet.
1	Chain, $\frac{1}{2}$ in., 7 ft. long.	124 ft	Rope, Manila, $1\frac{1}{4}$ in.
1	Clamp, Rivet.	675 ft.	Rope, Manila, 1 in., 5 pieces.
1	Chisel, Hand.	2	Lashings, 15 ft.
1	Dolly, Timber.	1	Stock and Dies, Blacksmith.
4	Drills, Twist, $\frac{11}{16}$ in.	1	Saw, Crosscut.
2	Files.	1	Saw, Hand.
2	Gouges, Handle.	5	Shovels, Short Handle
1	Hacksaw and Blades.	1	Shovels, Long Handle
3	Hammers, 7 lb.	1	Square.
3	Hammers, Claw.	1	Wrench, Bridge, $\frac{3}{4}$ in.
1	Hammer, Machine.	6	Wrench, Bridge, $\frac{5}{8}$ in.
3	Handles, 30 in.	2	Wrench, Bridge, $\frac{1}{2}$ in.
1	Jack Screw, 12 in.	1	Wrench, Stillson, 10 in
1	Level.	1	Wrench, Monkey, 12 in.
		4	Wheel Barrows.

ERECTION OF TRUSS BRIDGES.—Truss bridge spans are usually erected on falsework. The truss may be erected by means of a traveler or a derrick traveler or a derrick car. The usual procedure where a traveler is used will be briefly described. After the falsework and traveler are ready, lay out the center lines of the trusses on the falsework and locate the positions of the panel points. At each panel point place the necessary blocking for camber. Then beginning at the fixed end place the pedestals in position and place the lower chords and the floorbeams and stringers in position and distribute the pins. If the floorbeams and stringers will be in the way they are not placed until they are needed. The traveler is run to the center of the bridge and the center panel on each side is erected. The upper chord section is hoisted and held a little above its final position; the posts are raised, the diagonals are put in place and the pins are driven, or with a riveted truss the joints are field bolted in about 50 per cent of the holes. The panel on the opposite side is then erected and the top lateral struts and bracing are put in place, the floorbeams and stringers are connected up and the lower laterals are put in place, so that the center tower is fully braced. Great care must be used in erecting the middle tower to see that it is in exactly the proper place. After the center panel is complete, the traveler is moved toward the fixed end, erecting the trusses one panel at a time. The traveler is then run back to the center and the roller end of the trusses are erected. After the span is all connected up and all connections are properly bolted up, the blocking is knocked out and the bridge is swung clear. The details of erection vary with the type of truss and local conditions and the above description is intended to merely give an idea of the procedure. Truss bridges may also be erected by starting the traveler at the fixed end.

Where a derrick car or a derrick traveler is used the erection is commonly started at the fixed end.

TABLE XII.

List of Erection Tools for the Erection of a Steel Mill Building 60 ft. by 150 ft. with Corrugated Steel Covering; 43 Tons Steel, 7 Tons Corrugated Steel. Minneapolis Steel & Machinery Co.

Quantity.	Name and Size of Tool.	Quantity.	Name and Size of Tool.
1	Axe, Hand.	1	Forge, Complete.
4	Bars, Crow.	1	Gin Pole.
4	Bars, Connecting.	4	Gouges, Handle.
1	Box, Tool.	1	Hack Saw and Blades.
2	Braces.	1	Hammer, Claw.
4	Brushes, Paint.	1	Hammer, Machine.
1	Block, Steel, Single, Wire Rope,	6	Handles, 30 in.
	10 in.	1	Man, Old.
1	Block, Steel, Double, Wire Rope,	2	Punches, Backing out.
	10 in.	6	Punches, Corrugated.
1	Block, Wood Snatch, 10 in.	20	Pins, Drift, $\frac{5}{8}$ in.
10	Block, Wood, Single Tackle, 8 in.	10	Pins, Drift, $\frac{3}{4}$ in.
8	Block, Wood, Double Tackle, 8 in.	1	Ratchet.
700 ft.	Cable, $\frac{1}{2}$ in., 3 pieces.	1,100 ft.	Rope, Manila, $\frac{3}{4}$ in., 8 pieces.
1	Chain, $\frac{5}{8}$ in., 3 ft. long.	4	Rope, Manila, Lashings.
1	Chain, $\frac{1}{2}$ in., 8 ft. long.	1	Stock and Dies, Blacksmith.
1	Chain, $\frac{5}{8}$ in., 9 ft. long.	3	Snaps, Rivet, $\frac{5}{8}$ in.
23	Clamps, Cable, $\frac{5}{8}$ in.	1	Saw, Hand.
7	Clamps, Cable, $\frac{1}{2}$ in.	1	Square.
2	Clamps, Rivet.	4	Shackles.
6	Chisels.	2	Snips, Corrugated.
3	Cutters.	1	Tongs, Blacksmith.
1	Crab, Small.	2	Tongs, Heater.
1	Dolly, Timber.	1	Tongs, Pick-up.
1	Dolly, Goose Neck, $\frac{5}{8}$ in.	1	Vise, Machinist.
1	Dolly, Straight, $\frac{5}{8}$ in.	15	Wrenches, Bridge, $\frac{3}{4}$ in.
1	Dolly, Spring, $\frac{5}{8}$ in.	20	Wrenches, Bridge, $\frac{5}{8}$ in.
3	Dolly, Corrugated Steel.	8	Wrenches, Bridge, $\frac{1}{2}$ in.
1	Dolly, Jam, $\frac{5}{8}$ in.	1	Wrenches, Bridge, $\frac{3}{8}$ in.
1	Drills, Twist, $\frac{13}{16}$ in.	2	Wrenches, Monkey.

In erecting the Municipal Bridge over the Mississippi River at St. Louis, sand boxes were used for camber blocking in the place of the usual timber camber blocking.

The threads of pins should be protected by pilot nuts and pilot points when driving. Details of standard pilot nuts are given in Table 99, Part II, and of standard pilotpoints in Table 100, Part II.

RIVETING.—Field rivets may be driven by hand or with pneumatic riveters. Before driving the rivets the parts to be riveted must be drawn up by means of erection bolts so that the holes are fully matched and the surfaces of the metal are so close together that the metal from the rivet will not flow out between the plates. The holes are brought in line and matched by the use of drift pins, Fig. 7 and Fig. 17; care should be used not to injure the metal with the drift pin. If the holes will not match they should be reamed. A gang for hand riveting consists of four men, (1) a rivet heater, (2) a bucker-up, (3) a rivet driver, and (4) a man to catch and enter the rivets, to assist in driving and to hold the rivet set (snap). The hot rivet is thrown by the rivet heater with rivet-pitching tongs, Fig. 11; the rivet is caught in a bucket or keg and is put into the rivet hole with the rivet-sticking tongs, Fig. 11. The rivet is then bucked-up with a dolly, Fig. 9 or Fig. 10, and is upset with a rivet hammer, Fig. 7. After the rivet is upset to fill the hole a rivet set (snap), Fig. 7, is held over the upset rivet and a few blows with the riveting hammer completes the work. Field rivets are ordered with enough stock to furnish metal to fill the hole and to form a perfect rivet head. If the rivet is too short, either the hole will not be filled or the rivet

head will be imperfect. If the rivet is too long the rivet set (snap) will force the metal out under the edge of the rivet set (snap) making a bad looking job. The rivet should be heated uniformly so that it will be upset for its entire length. Riveters prefer to use rivets with scant stock so that

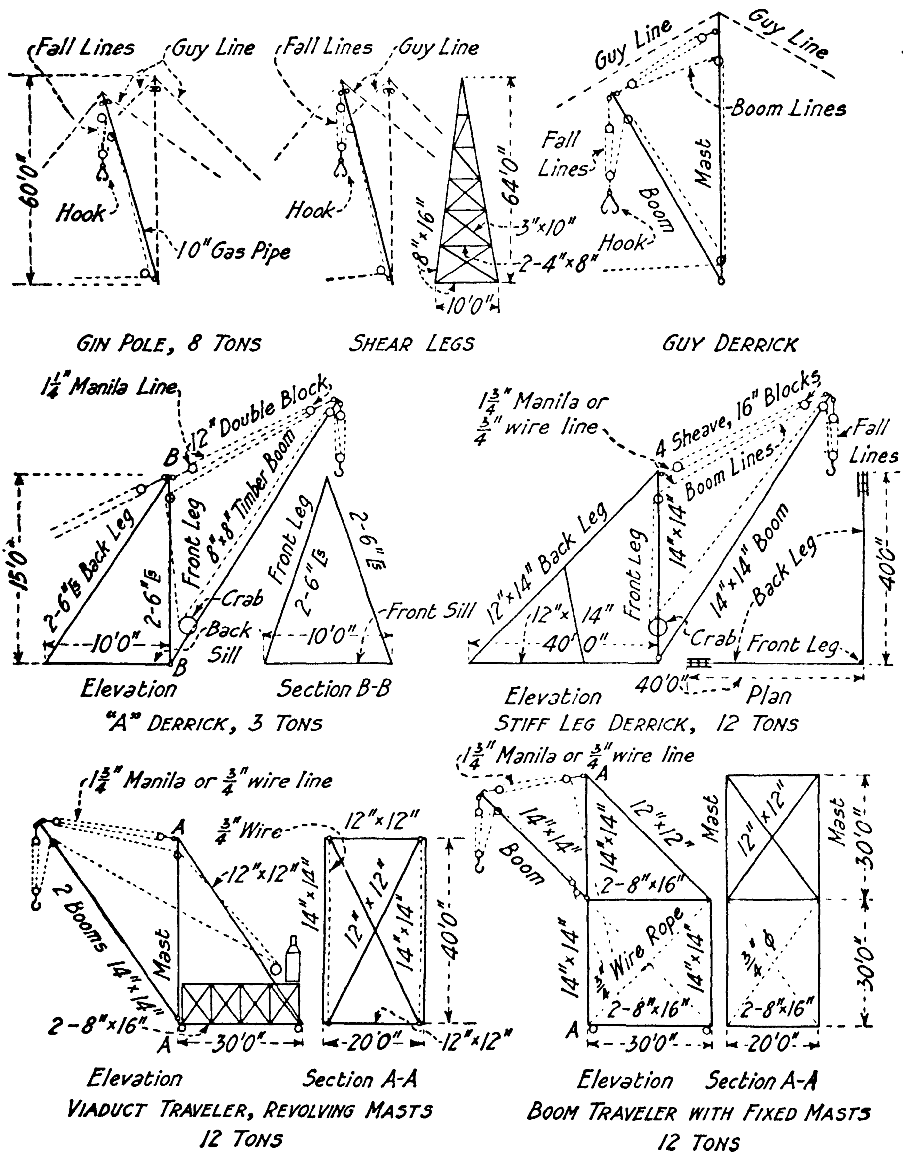


FIG. 18. DERRICKS AND TRAVELERS.

the rivet can be upset and a perfect head formed with little labor. To drive a rivet properly the rivet should be upset by striking it squarely on the end, as side blows will upset the rivet without filling the hole.

Where compressed air is available a pneumatic field riveter is used for driving rivets. Pneumatic field riveters are of two types: (*a*) jaw riveters that buck-up the rivet and form the head as in shop riveters; and (*b*) a pneumatic gun that is held against the rivet by the riveter, the rivet being bucked-up with a dolly as in hand riveting or with a pneumatic dolly. The pneumatic gun

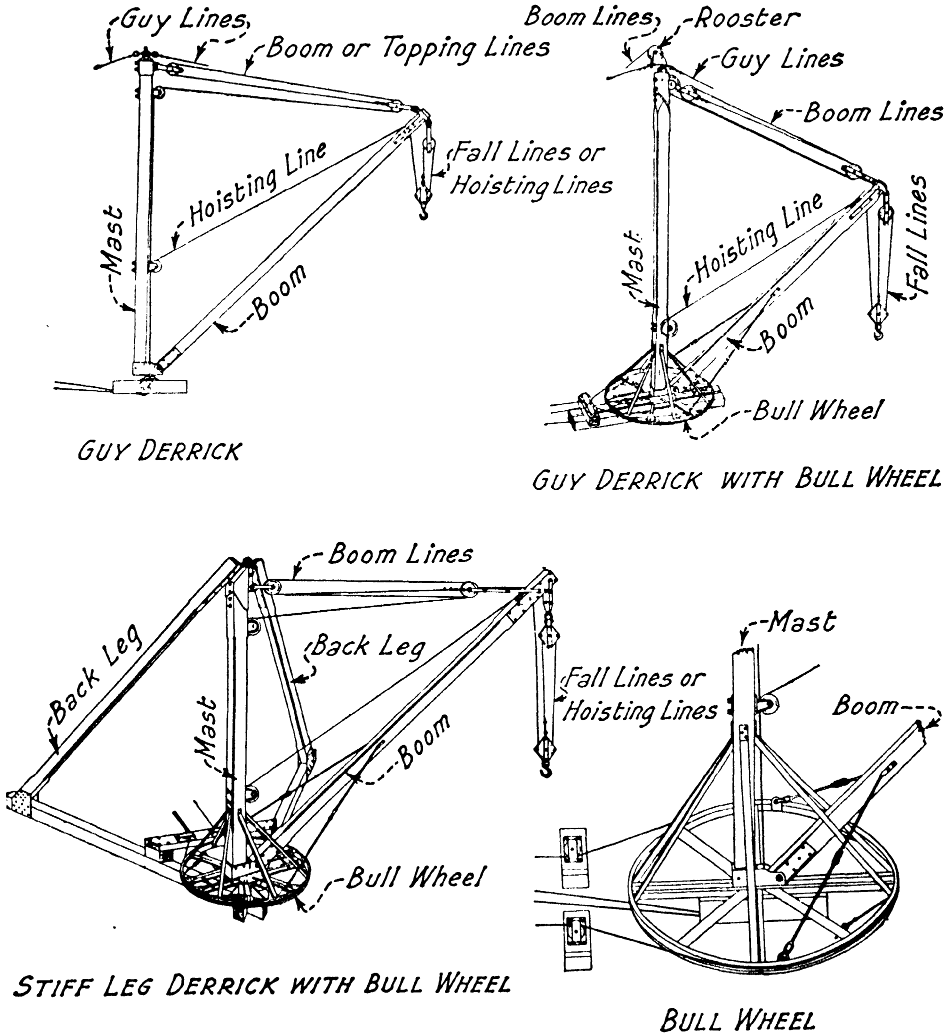


FIG. 19. DETAILS OF DERRICKS.

is more convenient and is commonly used. A rivet snap is used in the air gun. Good rivets can be driven by hand, but the work of the pneumatic riveter is more uniform and most specifications for erection of structural steel call for its use. Several railroad bridge specifications now require that hand driven field rivets be calculated for only four-fifths of the allowable stresses on machine driven field rivets. While more rivets can be driven with an air gun than by hand, the added expense for air makes the cost of driving nearly the same as for hand driven rivets.

Dollys for bucking-up rivets are made in many forms to suit the different conditions. Straight, goose-neck, bent, heel and club dollys are shown in Fig. 9, a ring dolly is shown in Fig. 10, and a corrugated iron dolly in Fig. 11. Dollys for use in erecting elevated tanks are shown in Fig. 16, and include the bar dolly, the heel dolly, the combination dolly, and the spring dolly.

DERRICKS AND TRAVELERS.—Derricks and travelers are made in many different forms. A few of the more common forms will be described.

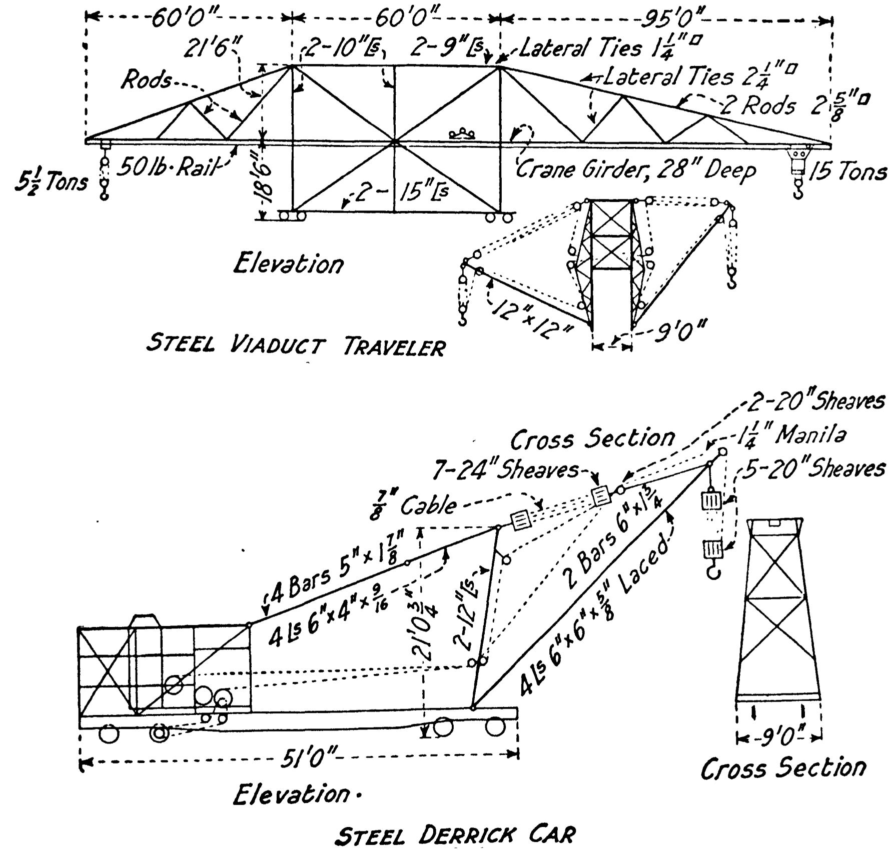


FIG. 20. DETAILS OF A VIADUCT TRAVELER AND A STEEL DERRICK CAR.

Gin Pole.—A gin pole, Fig. 18, is a timber or steel mast with four guys and a block at the top through which the hoist line leads to a crab bolted near the bottom, or the hoist line may run to the hoisting engine. The foot of a gin pole is supported by timbers which are shifted with bars or on rollers. The gin pole should not be inclined more than a few degrees from the vertical, and care must be used to prevent the bottom from kicking out with heavy loads. Gin poles may be made of timber, gas pipe, or may be built structural steel masts. Gin poles are not commonly made longer than 40 to 60 ft., but a trussed gin pole 120 ft. long has been used for erecting elevated towers. The mast of a gin pole may be built up so that only two guys are necessary, resulting in "shear legs" as in Fig. 18.

Each guy is fastened at its lower end to a "deadman" (a timber, or log, or beam buried in the ground).

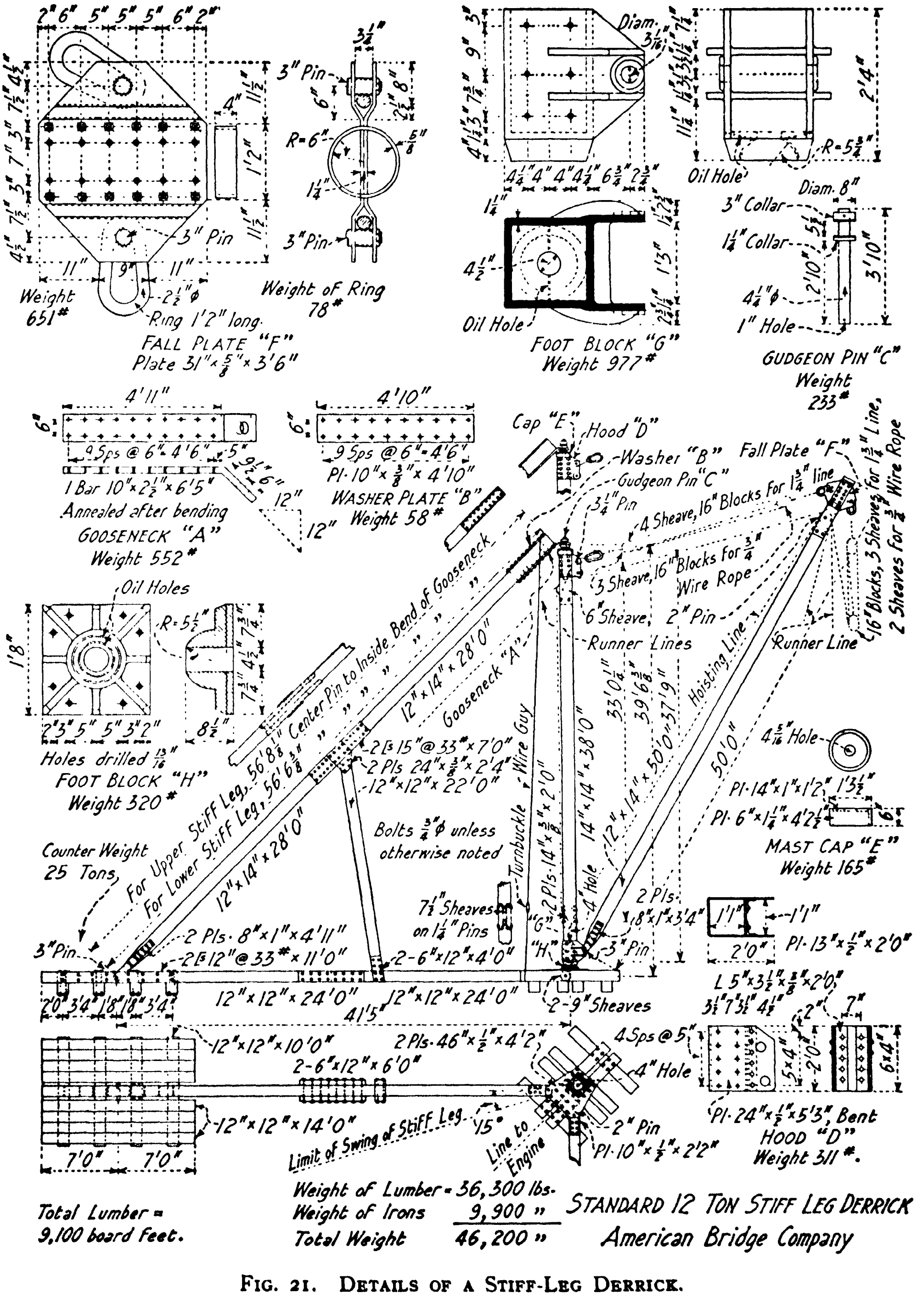


FIG. 21. DETAILS OF A STIFF-LEG DERRICK.

Guy Derricks.—A guy derrick, Fig. 18 and Fig. 19, has a vertical mast guyed with three or more guy lines, and has a boom which carries blocks and a fall line on the upper end. The boom is raised and lowered with rigging called "topping lines" or "boom lines." The load is raised by rigging called "fall lines" or "falls." The hoisting line may be run down the boom to a crab or to the hoisting engine, or the hoisting line may be run through a "rooster" placed on top of the mast and then to the hoisting engine. Guy derricks may be swung in a full circle, either by hand or by means of a bull wheel operated by a line from the hoisting engine.

"A" Derrick.—The "A" derrick or "Jinniwink" derrick is shown in Fig. 18. "A" derricks are used for light hoisting up to three to five tons. The "A" derrick is a simple form of the stiff-leg derrick.

Stiff-Leg Derrick.—The stiff-leg derrick has a mast braced by "A" frames set at right angles to each other, Fig. 18 and Fig. 19. The loads may be lifted and the boom raised and lowered by means of a crab or by a hoisting engine. The stiff-leg derrick has a free swing of about 240 degrees. The mast may be turned by hand or by means of a bull wheel operated by a line from the hoisting engine. Details of a 12-ton timber stiff-leg derrick are shown in Fig. 21. Stiff-leg derricks of large capacity are now commonly made of structural steel. Details of a steel stiff-leg derrick are given in Fig. 29.

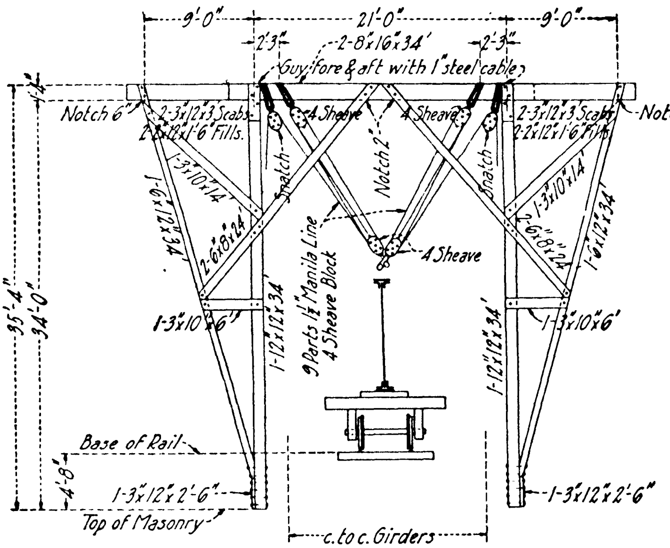


FIG. 22. DETAILS OF A GALLOWS FRAME. AMERICAN BRIDGE COMPANY.

Boom Travelers.—The mast of a derrick may be supported by the framework of a traveler, Fig. 18. The traveler may be made one or several stories in height. The booms may swing or may be fixed to raise and lower in one plane, and may be used single or in pairs. Boom travelers are commonly used in erecting train sheds, and structural steel buildings. Details of a steel boom traveler are given in Fig. 28 and Fig. 29.

Viaduct Travelers.—An overhang traveler for erecting a high steel viaduct is shown in Fig. 20.

Gallows Frame.—A gallows frame or a transverse bent as shown in Fig. 22, is used for erecting plate or riveted girders. The gallows frame is guyed fore and aft with steel cables. Gallows frames are commonly used in pairs or a gallows frame is used with a stiff-leg derrick.

Through or Gantry Travelers.—A through or gantry traveler consists of two or three transverse bents or "gallows frames" braced longitudinally and is carried on a track supported on the falsework and placed outside of the trusses. The traveler has a clearance such that it can be

TABLE XIII.

Bill of Timber in Traveler, Fig. 24.

No.	Cross Section, In.	Length, Ft-In.		No.	Cross Section, In.	Length, Ft-In.	
5	10 × 12	28–0	Hoisting beams.	4	4 × 8	18–0	Platform cut to 9 ft.
4	12 × 12	38–0	Longitudinal.	4	6 × 12	38–0	Sills.
2	8 × 16	44–0	Caps.	2	8 × 12	32–0	Sheave beams.
2	8 × 8	24–0	Chord.	10	4 × 8	36–0	Longitudinals.
4	8 × 10	30–0	Legs.	4	6 × 8	36–0	Platform.
4	8 × 10	24–0	Legs.	10	3 × 8	36–0	Platform plank.
4	6 × 8	32–0	Legs batter.	1	6 × 10	20–0	Blocks cut to 2 ft.
4	6 × 8	22–0	Legs.	4	6 × 10	28–0	Side braces.
8	4 × 8	26–0	Web braces.	4	6 × 10	30–0	Side braces.
4	3 × 8	16–0	Web braces.	2	4 × 6	16–0	Fillers cut to 8 ft.
4	3 × 8	14–0	Web braces.	2	4 × 6	14–0	Fillers.
4	3 × 8	12–0	Web braces.	1	3 × 8	12–0	Leg brace.
1	3 × 8	20–0	Web braces cut to 10 ft.	2	6 × 12	16–0	Fillers cut to 2 ft.
2	3 × 8	18–0	Leg braces cut to 9 ft.	2	8 × 10	16–0	Trucks cut to 8 in. × 9 in. × 4 ft.
2	3 × 8	20–0	Leg braces cut to 10 ft.				
2	3 × 8	12–0	Leg braces cut to 6 ft.	1	1 × 6	16–0	Fillers.
4	3 × 8	18–0	Leg braces platform.	4	3 × 8	20–0	Chord cut to 10 ft.
8	3 × 10	12–0	Leg splices cut to 6 ft.	2	3 × 8	22–0	Leg brace cut to 11 ft.
8	3 × 8	12–0	Leg splices cut to 6 ft.	1	3 × 8	18–0	Leg brace cut to 4 ft. 6 in.
8	3 × 6	12–0	Leg splices cut to 6 ft.	4	2 × 4	38–0	Sliding beam.

TABLE XIV.

Bill of Bolts in Traveler, Fig. 24.

No.	Diameter, In.	Length, Ft-In.
20	$\frac{3}{4}$	1–10
135	$\frac{3}{4}$	1–8
100	$\frac{3}{4}$	1–6
160	$\frac{3}{4}$	1–4
150	$\frac{3}{4}$	1–2
100	$\frac{3}{4}$	1–0
20	$\frac{3}{4}$	0–10
10	$\frac{3}{4}$	0–8
10	$\frac{3}{4}$	2–0
10	$1\frac{1}{4}$	1–4

TABLE XV.

Bill of Irons in Traveler, Fig. 24.

No.	Name.	Dimensions.
10	Sheave Chocks....	$10\frac{1}{4}$ in. Block Sheave.
4	Bent Bars........	3 in. × $\frac{1}{2}$ in. × 2 ft. 9 in.
4	Bent Bars........	3 in. × $\frac{1}{2}$ in. × 3 ft. 5 in.
2	Bent Bars........	3 in. × $\frac{1}{2}$ in. × 2 ft. 0 in.
2	Bent Bars........	3 in. × $\frac{1}{2}$ in. × 2 ft. 0 in.
16	Scabs............	3 in. × 1 in. × 1 ft. 10 in.
8	Rods............	$1\frac{1}{4}$ in. diameter × 9 ft. 2 in.
4	Traveler Wheels...	14 in. diameter, 3 in. shaft.
8	Wheel Boxes......	....
2	Rods............	$1\frac{1}{4}$ in. diameter × 6 ft. 6 in.
2	Rods............	$1\frac{1}{4}$ in. diameter × 3 ft. 6 in.

run past the completed bridge or structure. Travelers may be made of timber or structural steel. Outline plans for four standard timber travelers designed by the American Bridge Company are given in Fig. 23, while the detail plans for traveler No. 1 are given in Fig. 24. The bill of lumber for traveler No. 1 is given in Table XIII; the bill of bolts is given in Table XIV, and the bill of irons in Table XV. Traveler No. 1 may be used for single track railway spans up to 250 ft.; traveler No. 3 for single track spans up to 175 ft.; traveler No. 2 for double track spans up to 175 ft.; and traveler No. 4 for double track spans up to 250 ft.

Derrick Cars.—Derrick cars with a capacity up to 75 tons are in common use. The derrick cars are usually self-contained and can move under their own power. The boom can be folded back over the car out of the way when not in use. A sketch of a derrick car is shown in Fig. 20.

FALSEWORK.—Falsework for the erection of bridges is built up of bents made of three or more posts or piles, braced transversely in the same manner as for permanent trestles. Framed bents are carried on mudsills, or on piles where the foundation is inadequate or where the falsework is in flowing water. Where piles can not be driven in running water or where there is danger

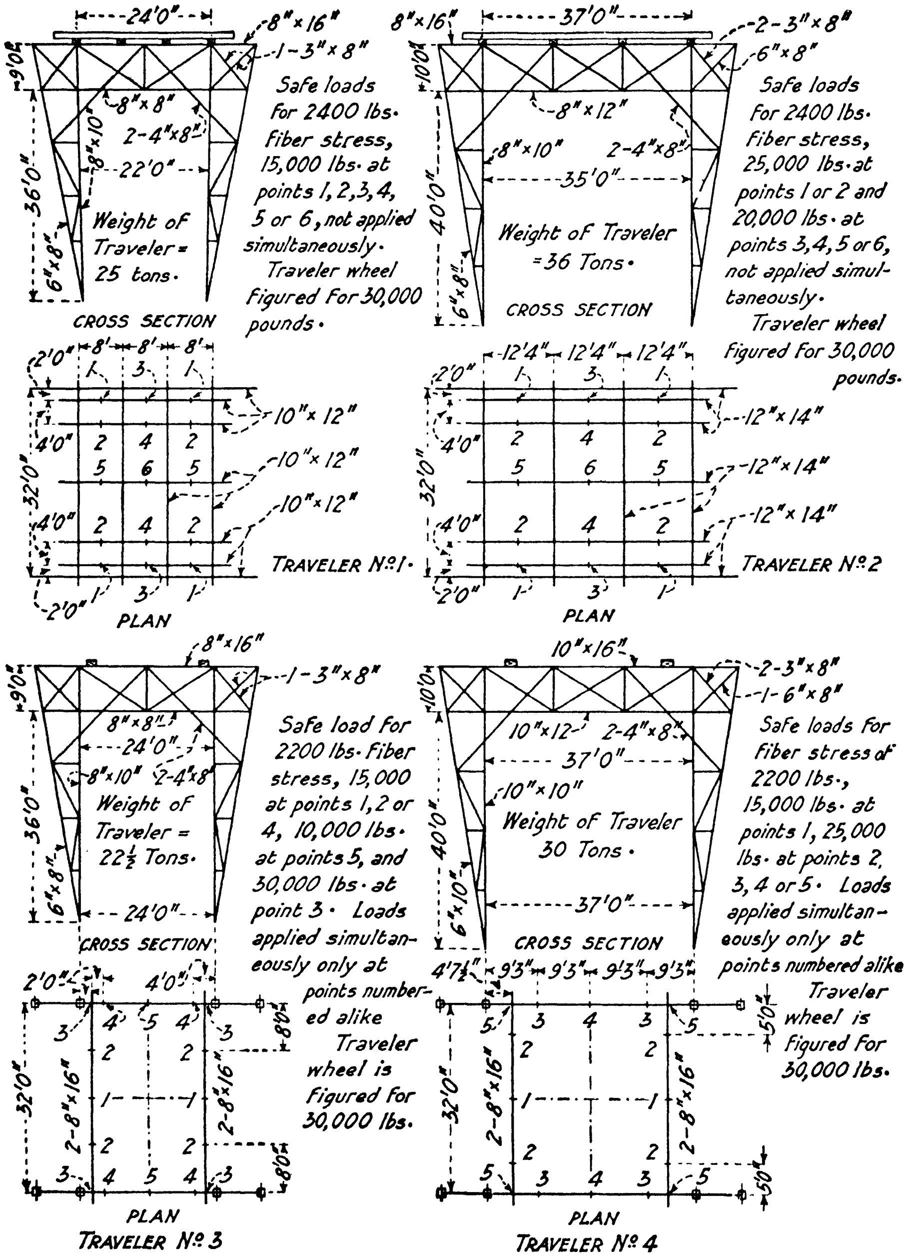


FIG. 23. STANDARD TIMBER TRAVELERS. AMERICAN BRIDGE COMPANY.

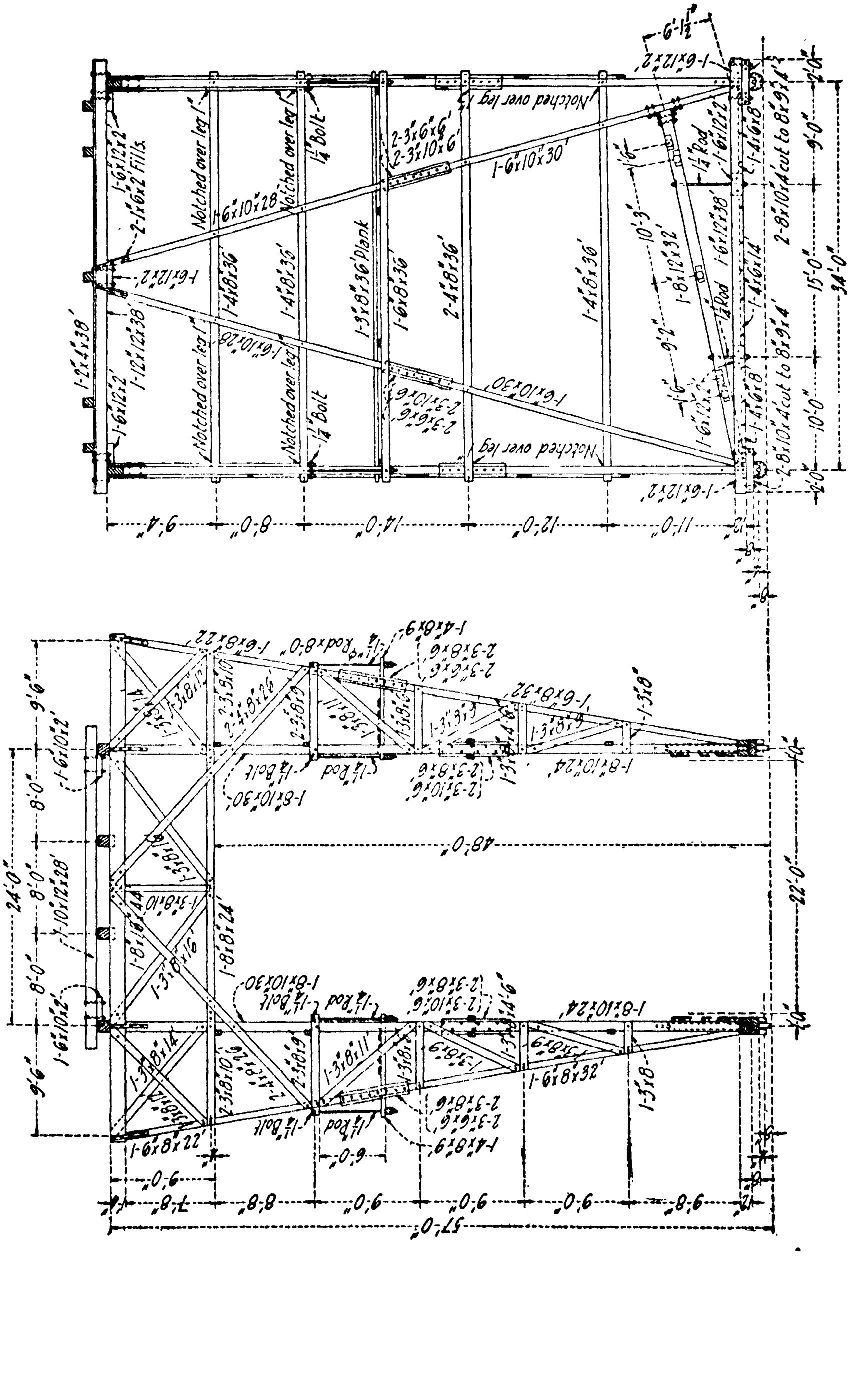


FIG. 24. THROUGH BRIDGE TRAVELER. AMERICAN BRIDGE COMPANY.
(Traveler No. 1 in Fig. 23.)

of flood, it may be necessary to use spread footings which are anchored in place. Where it is practicable to obtain piles of sufficient length they may be used for the full height of the falsework. The timber used in building falsework should be sound, strong, free from defects that will affect its strength or interfere with its use. Since the structure is temporary, durability is not an important element in selecting timber for falsework unless it is to be used several times.

For examples of timber trestles, see Chapter VII.

Plans of typical four-legged falsework as used by the American Bridge Company are shown in Fig. 25. When trains are to be carried and 2–8 in. × 16 in. stringers are used under each rail, bents must not be spaced over 18 ft. centers for the falsework as shown.

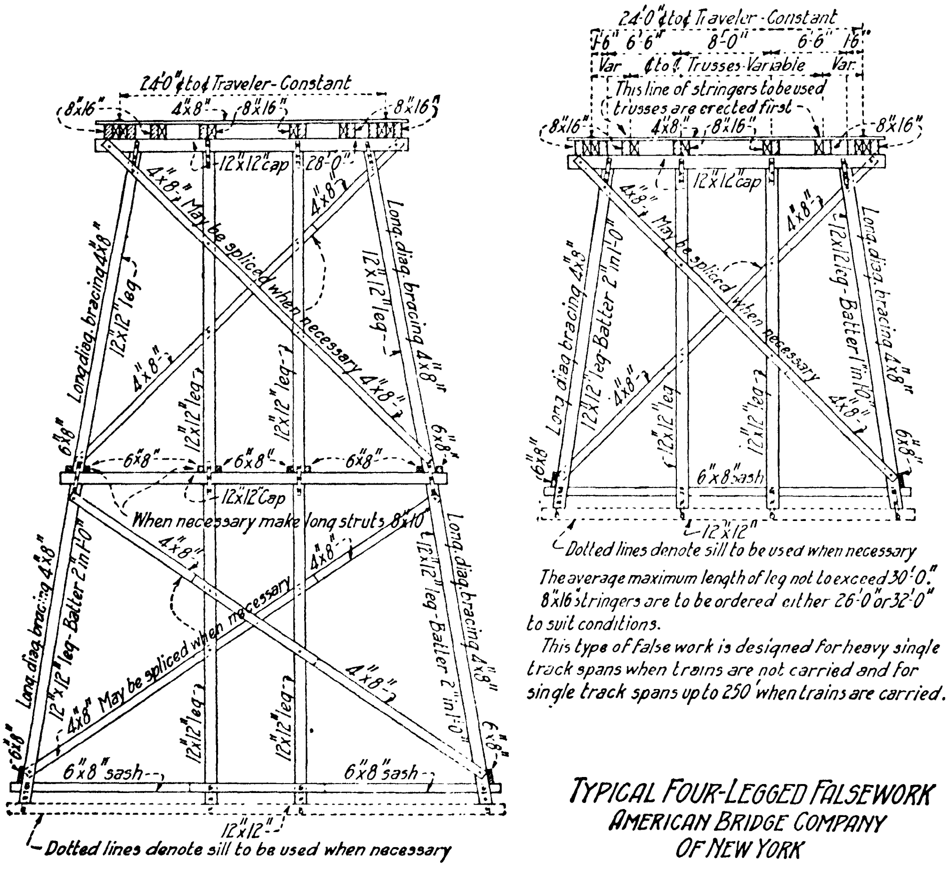


FIG. 25.

Piles.—Timber piles may be driven with a drop hammer, Fig. 26, or with a steam hammer. A spool roller pile driver with a drop hammer is shown in Fig. 26. The hammer is raised to the top of the leads by the hoisting engine; the hammer is then permitted to fall on the top of the pile, dragging the hoisting rope down with it. The force of the blow of the hammer depends upon the weight of the hammer, the height of free fall, and the resistance of the hammer in the leads. By catching the hammer as it descends the operator can cushion the blow so that the safe bearing power of a pile as calculated from the penetration may be very misleading.

Details of a pile driver are given in Fig. 27.

The safe load on piles may be calculated by the Engineering News formula

$$P = \frac{2W \cdot h}{s + 1} \qquad (1)$$

where P = safe load on the pile in tons;
W = weight of hammer in tons;
h = height of free fall of hammer in ft.;
s = average penetration of the pile for last six blows.

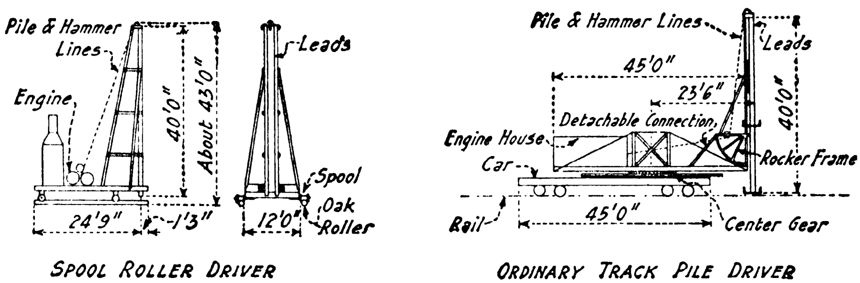


FIG. 26. TYPES OF PILE DRIVERS.

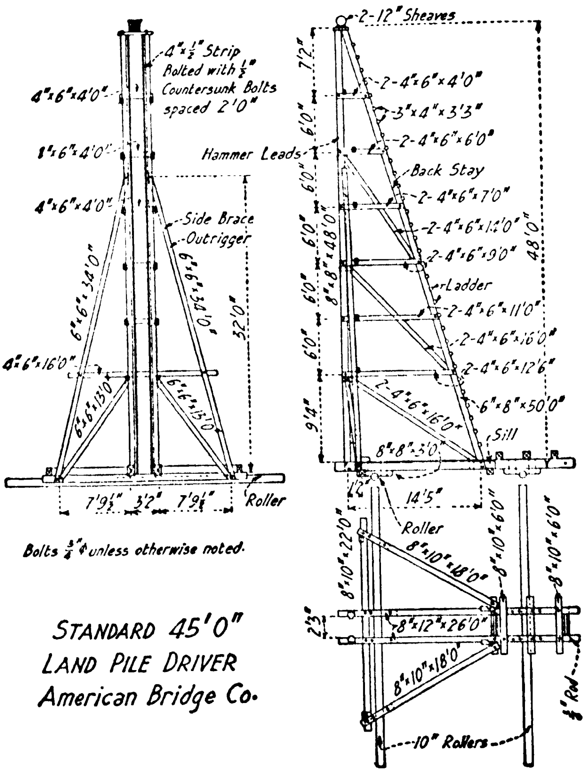


FIG. 27. DETAILS OF STANDARD PILE DRIVER.
AMERICAN BRIDGE COMPANY.

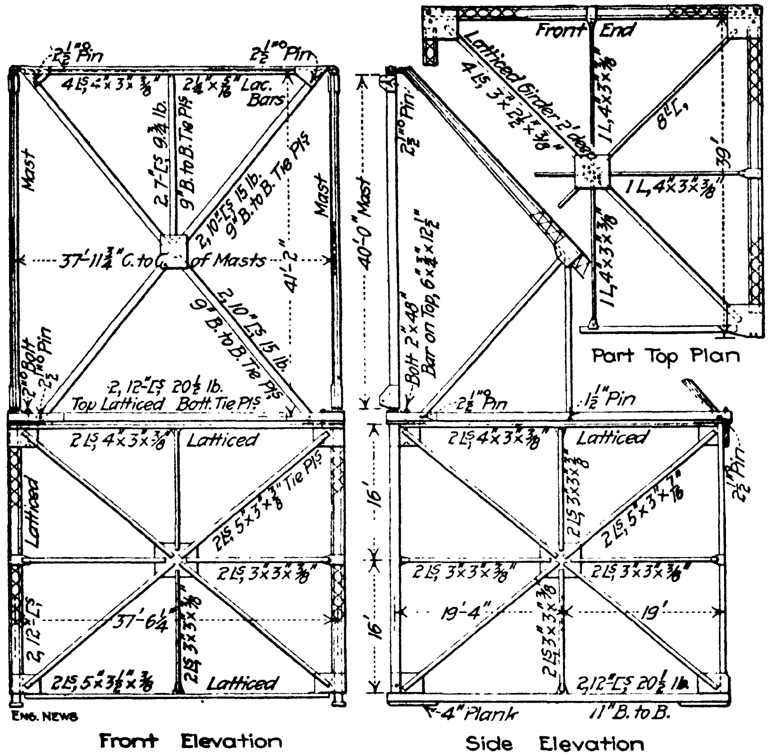


FIG. 28. TRAVELER USED IN ERECTION OF ARMORY, UNIVERSITY OF ILLINOIS.

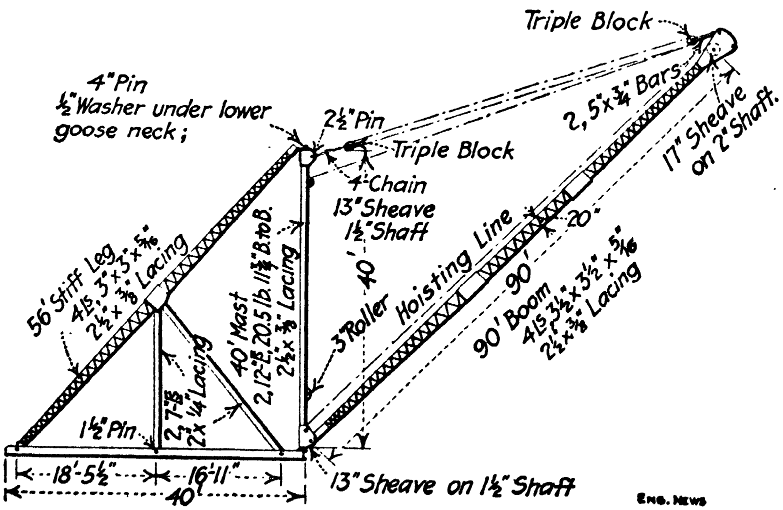


FIG. 29. STIFF-LEG DERRICK USED ON ERECTION TRAVELER FOR ERECTION OF ARMORY, UNIVERSITY OF ILLINOIS. (Two of these derricks were used on front of traveler.)

Piles should have a penetration of not less than 10 ft. in hard material and not less than 20 ft. in soft material. For a steam hammer unity in the denominator in (1) should be replaced by $\frac{1}{10}$.

The following specification is commonly used for piles for heavy falsework.

All piles are to be spruce, yellow pine or oak, not less than 9 in. in diameter at the point and not more than 14 in. in diameter at the butt. Piles are to be straight and sound, and free from defects affecting their strength or durability. Piles are to be driven into hard bottom until they do not move more than ½ in. under the blow of a hammer weighing 2,000 lb. and falling 25 ft.

For specifications for falsework piles, see Chapter VII.

A track pile driver is shown in Fig. 26.

Design of Falsework.—Falsework should be designed to carry the necessary loads. Where the falsework is required to carry traffic it should be designed for the same allowable stresses as are permitted for timber trestles and bridges, Table V, Chapter VII. Where the falsework does not carry traffic the allowable stresses may be fifty per cent in excess of those permitted for permanent structures. Care should be used in the design to prevent crushing of timber across the grain. For details of timber trestles see Chapter VII.

Traveler for Erection of Armory.*—The new armory for the University of Illinois is 276 ft. by 420 ft. in plan, the main drill hall being covered by three-hinged arches with a span 206 ft. centers of end pins, a center height of 94 ft. 3 in., and are spaced 26 ft. 6 in. The arches have a horizontal tie of two 4 in. × ⅝ in. bars, and are braced together in pairs.

Each arch was shipped in eight segments, and the four sections for each half of the arch were assembled and riveted up in horizontal position on the ground close to their final positions. One side of the arch was then lifted into a vertical plane by a two-boom traveler, and its lower end was fitted into the shoe and the shoe pin driven. The truss was then lowered on this pin until its head rested on the ground, the arch segment being supported by guys at the sides. The opposite segment of the arch was then raised and adjusted in the same way. The traveler was then placed at the center of the arch, and the hoisting lines of the two booms were attached near the ends of the two half-arches, which were then raised, the lower ends rotating on the shoe pins. The arch was then held while the center pin was driven and the purlins were placed connecting it to the adjacent arch.

The traveler, Fig. 28, consisted of a steel tower about 40 ft. square and 33 ft. high to the working deck. On this deck were two 40-ft. masts with A-frames, each carrying a 90-ft. boom, so that the top of the boom could reach about 20 ft. above the top of the arches, the maximum height from the ground to the hoisting block being 125 ft.

The traveler was supported on wood rollers on tracks of 16 × 16 in. timbers about 40 ft. apart. The upper part of the traveler was composed of two stiff-leg derricks of the type shown in Fig. 29, with one stiff-leg and one sill removed from each, the masts being stepped on the traveler frame and connected by bracing as shown. Each derrick had a lifting capacity of 15 tons, and was operated by an engine of 8 H. P., the two engines being placed on a platform on the lower sills of the traveler about 2 ft. from the ground.

INSTRUCTIONS FOR THE ERECTION OF STRUCTURAL STEEL.—The McClintic-Marshall Construction Co. has issued the following instructions to foremen.

In Order to Avoid Accidents, as Far as Possible, be Guided by the Following:

1. **See that Your Equipment is Sufficiently Strong.**—It is your duty to see that the equipment and tools you use for each part of the work are sufficiently strong to handle the same safely.

You should see that the derricks you use are amply strong for the loads to be lifted. The goose neck and gudgeon pin are the critical points of a derrick. If you have any doubt about the strength of the goose neck, provide heavy wire guys from gudgeon pin to sill at base of stiff legs. Don't lift a ten ton load on a five ton derrick. The same thing applies to gin poles and travelers. Don't overload your equipment and don't run any chances where life is endangered. Be careful not to lift any but a light load on a derrick if the length of the boom exceeds seventy times the least width or thickness of the boom; that is, if your boom is 12 in. × 14 in. the least width is 12 in., you should not lift a heavy load on this boom if it is more than seventy feet in length.

* Engineering News, Dec. 11, 1913. The structural steel was fabricated and erected and the traveler was designed by the Morava Construction Co., Chicago, Illinois.

See that travelers are well and carefully framed and erected, well braced and capable of withstanding the greatest wind, and shocks from heaviest loads that are to be lifted.

See that the hooks, shackles and beckets on your blocks are amply strong, and don't allow a gate block to be used without it being closed and hooked. Also see that your cables and chains, as well as the rings and hooks in the same, are amply strong for the loads to be lifted.

Do not use old or worn line when there is any danger to men or material by so doing. Cut out the use of manila line whenever possible. When you are obliged to use it be sure it is amply strong. Use steel cable whenever possible, as it is safer, will last longer and is cheaper in the long run. Be sure that the guy cables for gin poles, derricks, etc., are of sufficient size to withstand the tension to come upon them. Also that the cables are securely fastened by means of a sufficient number of good, strong clamps well fastened, and also that dead men or other anchorages are ample, and watch them when lifting heavy loads to see that guys do not cut dead men in two. Keep gin pole guys as near at right angles to each other as possible, when only four are used.

You should be careful to see that the gas pipe or wooden scaffold you use is of proper size and strength for the span and loads. If there is any question about the strength, test the same by applying several times the load that will come upon it. See that plank you use for scaffolding, etc., is the right kind of wood, preferably white or yellow pine, free from knots and shakes and plenty strong, watching to see that it is thick enough for the span on which it is used.

Do not put heavy loads on light push cars. The frame is not only liable to crush but the shafts, boxes or wheels may bend or break, upsetting the load and injuring the men.

2. **See That Your Equipment is in Order.**—In setting up your derricks see that they are plumb, properly guyed and that the splices are brought into contact and bolted with tight-fitting bolts. See that the goose-necks fit gudgeon pin closely and are not cracked or bent and that the top of stiff-leg is tied down from the goose-neck to the sill to prevent lifting tendency. If the timbers in the mast, boom, stiff-legs or sills are rotten, knotty or wind shaken, do not use them. See that your gudgeon pin and pintle casting are well fastened to the mast, and if the mast is of wood that the wood is not rotten or worn at these points.

You should see that all leads are as straight and direct as possible, as failure to provide good leads reduces the efficiency of your power and equipment, as well as producing heavy wear on the lines and is a frequent cause of accidents. Particular care should be exercised in securing good leads for wire cable on account of liability of breaking the individual wire strands by sharp bends or indirect leads. A broken individual wire is liable to lie across and cut the other wires of the cable. When you use a wooden traveler see that the timbers are all in good condition and that it is erected plumb and square and the joints are properly and securely bolted. *More accidents occur from the use of wooden derricks and wooden travelers than from any other cause*, and for this reason extreme care should be exercised to see that they are in good condition before using them. When a traveler is used, see that it is properly erected and thoroughly bolted and all sway and bracing rods tightened.

Do not use an iron gin pole if the sections are bent or dented seriously, or the splices do not clamp the pole tightly and securely. Do not use a wooden gin pole unless the timber is in good condition, well spliced with good long splices securely bolted.

See that your hoisting engine is in good order; that the shafts are not bent, the dogs, clutches and brakes, including the friction, are in good condition and working order. The lever controlling the winch heads should be straight and when thrown in should engage the ratchet fully. See that winch head cannot slip off shaft. See that the boilers are cleaned frequently and kept in good condition.

You should be particular to see that gas pipe scaffolding is not rusted on the inside and that it is fastened so that it cannot roll or turn. Do not use any plank or timber for scaffolding that is knotty, rotten or weather cracked, and allow no man to work on scaffold plank laid loose on the supports. The plank should be fixed so that they cannot move or slide endwise, by using drop bolts.

All cables should be in good condition and kept oiled or greased so that they will not rust; if they are not in good condition, do not use them. All guy cables should be securely fastened by means of a sufficient number of good clamps.

See that your chains and the rings and hooks in the same are not worn, cracked or bent out of shape and that they are annealed at least once every three months in an annealing furnace, if you are near one, or otherwise anneal them yourself by laying them down in a straight line and building a good sized wood fire over them, heating slowly to a cherry red, then cover over thoroughly with ashes and heated dry dirt leaving them to cool slowly in the ashes and dirt. In laying the chains down in a straight line do not lay one chain on top of another. Be particular to see that the covering is ample so that air or moisture cannot cool the chains quickly or partially. This annealing should be done on Saturday and chains not disturbed until Monday. Chains used frequently every day should be annealed once a month.

See that your blocks are in good order and that the beckets, shackles and hooks are not bent, cracked or out of shape, and that faces of blocks are in good condition, also that the sheaves are not cracked or the flanges broken.

See that all button sets (rivet sets) are fastened to the air hammers.

3. **See that Your Equipment and Tools are Properly Used.**—In using a locomotive crane be sure that your track is properly ballasted and level and the rails well spiked down. *Do not lift a load sideways when the locomotive crane is standing on a curve, without using extra care. Use your outriggers and rail clamps when lifting a heavy load.*

The loads that a locomotive crane is capable of handling safely for each radius are plainly marked on the crane; don't attempt to lift heavier loads with the crane.

See that the booms of locomotive cranes, derrick cars or derricks, are in first class condition. If the boom (or flanges of the boom) has been injured or bent, don't use it, but replace the broken or bent part with new material. Don't attempt to straighten it, as the material in all probability has been injured, and will break or collapse sooner or later.

A locomotive crane is a useful, but dangerous piece of equipment, for this reason the greatest possible care should be exercised in handling the same. *Don't allow any man on the car or crane cab, except the craneman, and keep workmen from under the boom.* Don't attempt to shift track with your crane standing on the same track, and don't attempt to lift a maximum load with the boom horizontal.

You must be especially careful in swinging boom sidewise or lifting loads sidewise with a derrick car as your car will upset unless you use outriggers or guys. · Don't run chances, but lift the load straight ahead wherever possible. See that the boom on the derrick car is tightly guyed at all times with wire rope running from end of boom to sides of car. Never use manila line for this purpose, as it will stretch and your boom will get away from you, upsetting the car. Use additional guys to end of boom when setting heavy loads.

In carrying loads with a locomotive crane or derrick car on a curve, be sure that the track is level and the outer rail not elevated as is customary with railroad track.

Be very careful in using a wooden boom extension or outriggers, that you do not lift too heavy loads. The increased length of the boom and the weight of extension reduce the lifting capacity considerably. Whenever possible, avoid the attachment of guy lines to railroad tracks, as numerous accidents have occurred by car running into the guys.

Hook onto sheets or bundles of small material so that they cannot slip out.

Don't allow men to carry glazed window sash on their shoulders when the wind is blowing.

See that gate blocks are securely fastened and that men do not stand in the "bite" of a line.

Do not use a light gate block when lifting heavy loads.

Lines should be run around two winch heads when making a heavy lift.

When you use a derrick keep the boom elevated above a horizontal line as far as possible, as generally the worst stress comes on the boom and mast as well as stiff-legs or guy lines when boom is in a horizontal position. A maximum load for the derrick should never be lifted with the boom in a horizontal position.

When you use a gin pole see that the splices are well bolted and the pole is properly guyed. Do not lean the pole too much when lifting a load or moving the pole and see that the foot of the pole cannot move or slip except when you desire to move it.

A number of accidents have occurred through the improper loading of push cars. See that the load is properly placed so that it cannot roll or tumble over, especially going around a curve. Do not allow your men to push on the side of the car with a top heavy load. They should push or pull from the ends of the piece.

When you lift a beam or girder use scissor dogs or cast steel girder hooks wherever possible, and if you are obliged to use either ordinary dogs or chains see that wooden blocks are used between the chain or dog and the flange to prevent the girder from slipping.

Avoid the use of chains except for lifting light loads. Where you have heavy loads to lift use cable slings, being careful to avoid sharp bends by using rounded wooden blocks between cable and load. Don't put too many parts of lashing into a hook as by doing so you are liable to open up the hook. See that exposed parts of dangerous machinery are properly covered.

4. **Be Orderly, Careful.**—See that your work is carried on in an orderly, careful manner.

See that material is unloaded and piled in an orderly, careful way so that it cannot fall, turn or be blown over.

Unless necessary, do not hoist any material to a structure until you are ready to put it into position and properly fasten it. In cases where you do hoist material to the structure before putting it in its final position, see that it is piled in an orderly way so that it cannot turn or roll over when a man steps on it.

Don't let tools or equipment such as bolts, nuts, drift pins, blocks, dolly bars, etc., lie around so that they can be knocked off the work or so that any one can fall over them. Keep everything orderly and in ship-shape and allow nothing to lie around.

5. **Be Vigilant.**—You must use vigilance and be on the job practically all the time to see that your men are carrying out your instructions; that tools and equipment are in fit condition for the work and that they are handling the work carefully and intelligently.

Be careful and insist on the men under you being careful, and do not allow any one who is reckless and careless to work for you.

Whenever any question as to the safety of equipment or tools or the work which you are erecting is brought to your attention by any of the men under you or others, investigate the same and satisfy yourself of the safety of the same before proceeding further. If you are satisfied the work, equipment or tools are not safe, put them in a safe condition immediately.

6. **See that Proper Instruction is Given Employees.**—Call attention of men to any dangerous conditions on the job so that they can be on the lookout. Your faithful attention to this matter is to the interest of employee and employer alike.

7. **Unfit Condition.**—You must see that every employe under you is in proper physical condition. They should be strong, temperate, clear-headed, with good eyesight, good hearing, and not lame or crippled.

Do not allow any man to go to work who has been drinking or drinks during working hours or who is sick or in unfit condition. A man's mind is not clear who is at all under the influence of liquor and thus endangers his own and fellow workmen's lives. Don't employ ignorant persons.

Don't employ any one under eighteen years of age and preferably no one under twenty-one. Those employed between the ages of eighteen and twenty-one should be strong, sober, healthy boys who desire to learn the business. You must secure a written permit from the parents of all boys under twenty-one years of age, authorizing you to employ them. Forms for this purpose will be sent you. The character of this business is such that a workman should be strong and sound in body, temperate in habits, clear and alert in mind, to avoid accidents.

8. **Use Judgment.**—You must use judgment in assigning men to do certain work and see that they are capable and experienced in the work to be done.

Signal men should be capable, experienced bridgemen, and should stand in a position where they can be seen by the men at the hoisting engine and those connecting the work. Signals should be clearly understood. Use none but good, careful, experienced locomotive cranemen, derrick car men, and men on winch heads.

Don't resort to expediency by allowing an inexperienced man to do the work where experience counts. Educate the men up to their work. Don't throw too much on inexperienced men all at once. You should see that the pusher and men use proper tools to do the work and handle same properly. Don't allow your men to work on crane runway when cranes are in motion. Don't allow men to work on scaffold that you would not work on yourself. Where there are heavy pieces to be lifted see if the weight is marked on the piece; if not, get the weight from the invoice and mark it on, calling pusher's attention to it.

9. **Do Not Allow Men to Work in Perilous Places.**—You must see that your men are not exposed to extremely hazardous conditions and that they are not allowed to work in extremely dangerous places.

Do not allow your men to work under loads and in places where there is imminent danger.

Be careful not to allow men to work on the roofs of buildings when there is frost, ice or snow on the same, without taking extreme precautions. The same applies to other steel structures.

10. **See That Workmen Obey Following Rules.**

a. **Don't Be Reckless.**—More accidents occur through recklessness than any other cause. Don't walk on rods. Don't ride a load. Don't ride on a locomotive crane.

b. **Don't Be Careless.**—Look where you step and be sure that on what you step is safe and secure. Don't step on ends of loose plank. Don't start to slide down a line unless you are sure the ends are fastened.

c. **Be Orderly.**—Do whatever you do in an orderly, careful manner. Pile material so that it cannot roll, fall, tumble, or be blown over. Don't let tools or equipment such as bolts, nuts, drift pins, blocks, dolly bars, etc., lie around so that they can be knocked off the work or so that any one can fall over them.

d. **Unfit Condition.**—Don't go to work if you have been drinking or do not feel well. If you are lame or have any defect in hearing or eyesight you should not work at this business as by so doing you endanger your own and fellow workmen's lives. If you are inexperienced in, or unsuited for the work to be done, don't undertake it.

e. **Be Vigilant.**—Watch what you are doing. Don't stand or work under a load. Don't go in the "bite" of a line nor stand in front of a snatch block. Don't work on or about a crane runway when the crane is in use unless there is a stop between you and the crane.

f. **Don't Use Unfit Tools.**—Be sure the tools and equipment you use are in good working order. If they are not, don't use them. Don't work with men who don't observe these rules.

GENERAL SPECIFICATIONS FOR THE ERECTION OF STEEL RAILWAY BRIDGES.

AMERICAN RAILWAY ENGINEERING ASSOCIATION.

For Fixed Spans Less Than 300 Feet in Length.

1923.

1. **Definitions of Terms.**—The term " Engineer " refers to the Chief Engineer of the Company or his subordinates in authority. The term " Inspector " refers to the Inspector or Inspectors representing the Company. The term " Company " refers to the Railway Company or Railroad Company party to the agreement. The term " Contractor " refers to the erection contractor party to the agreement.

2. **Work to Be Done.**—The Contractor shall erect the metal work, make all connections and adjustments, remove the old structure and falsework, and do all work required to complete the bridge or bridges, as covered by the agreement, in accordance with the plans and these specifications.

3. **Drawings to Govern.**—Where the drawings and the specifications differ, the drawings shall govern.

4. **Plant.**—The Contractor shall provide all tools, machinery, and appliances, including drift pins and fitting up bolts, necessary for the expeditious handling of the work. The Contractor shall protect the Company against claims on account of patented devices or parts used by him on the work.

5. **Plans.**—The Company will furnish complete detail plans for the bridge or bridges to be erected, including shop details, camber diagrams, erection diagrams, match-marking diagrams, list of field rivets and bolts, and copy of shipping statements showing a full list of parts and weights.

6. **Delivery of Materials.**—The Contractor shall receive all materials entering into the finished structure, free of charge at the place designated, loaded or unloaded, as specified in the information given bidders.

7. **Handling and Storing Materials.**—The Contractor shall unload material promptly upon delivery, otherwise he shall be responsible for demurrage charges. Stored material shall be piled securely outside the tracks, and no material shall be placed closer than six feet to the near rail. Material shall be placed on skids, above the ground. It shall be kept clean and properly drained. Girders and beams shall be placed upright and shored. Long members, such as columns and chords, shall be supported on skids placed near enough together to prevent injury from deflection. The Contractor shall check all material turned over to him against shipping lists and report promptly in writing any shortage or injury discovered. He will be held responsible for the loss of any material while in his care, or for any damage resulting from his work.

8. **Falsework.**—Unless otherwise provided, the Contractor shall prepare and submit to the Engineer for approval, plans for falsework, or for changes in an existing structure necessary for maintaining traffic. The falsework shall be properly designed and substantially constructed and maintained for the loads which will come upon it. Approval of the Contractor's plans shall not be considered as relieving the Contractor of any responsibility. Temporary structures or falsework placed by the Company, if suitable, may be used by the Contractor.

9. **Masonry.**—The Company will construct the masonry to correct lines and elevations, and will establish the lines and elevations required by the Contractor for setting the steel.

10. **Bearings and Anchorage.**—Bed plates, bolsters, and shoes shall be set level in exact position. They shall be given full and even bearing by setting them on a layer of Portland cement mortar or dry cement, or by tightly ramming in rust cement after blocking them accurately in position, as directed by the Engineer.

11. The Contractor shall drill the holes and set the anchor bolts, except where the bolts are built into the masonry. The bolts shall be set accurately and fixed with Portland cement grout completely filling the holes.

12. **Methods and Equipment.**—Before starting work, the Contractor shall advise the Engineer fully as to the method he proposes to follow, and the amount and character of equipment he proposes to use, which shall be subject to the approval of the Engineer. The approval of the Engineer shall not be considered as relieving the Contractor of the responsibility for the safety

of his method or equipment or from carrying out the work in full accordance with the plans and specifications. No work shall be done without the sanction of the Engineer.

13. **Assembling Steel.**—All parts shall be accurately assembled as shown on the plans and any match-marks carefully followed. The material shall be carefully handled so that no parts will be bent, broken or otherwise damaged. Hammering which will injure or distort the members will not be permitted. Bearing surfaces and surfaces to be in permanent contact shall be cleaned just before the members are assembled. Unless erected by the cantilever method, truss spans shall be erected on blocking so placed as to give the trusses proper camber until all tension chord splices are fully riveted and all other truss connections pinned and bolted. Rivets in splices of butt joints in compression members shall not be driven until the span has been swung. Splices and field connections shall have one-half of the holes filled with bolts and cylindrical erection pins (half bolts and half pins) before riveting. Splices and connections carrying traffic during erection shall have three-fourths of the holes so filled.

Fitting up bolts shall be of the same nominal diameter as the rivets, and the cylindrical erection pins shall be $\frac{1}{32}$-inch larger.

14. **Riveting.**—Riveting preferably shall be done with pneumatic riveters and buckers. Rivets larger than $\frac{7}{8}$-inch in diameter shall not be driven by hand. Connections shall be accurately and securely fitted up before the rivets are driven. Light drifting will be permitted to draw the parts together, but drifting to match unfair holes will not be permitted. Unfair holes shall be reamed or drilled. Rivets shall be heated to a light cherry color, and in driving shall be upset to completely fill the holes. Heads shall be full and symmetrical, and concentric with the shank, and shall have full bearing all around. They shall be of the same shape and size as the heads of the shop rivets. Rivets shall be tight and shall grip the connected parts securely together. No recupping or caulking will be permitted. Rivets shall not be overheated or burned. In removing rivets, the surrounding metal shall not be injured; if necessary, rivets shall be drilled out. Cup faced dollies, fitting the head closely to insure good bearing, shall be used.

15. **Bolted Connections.**—In bolted connections, bolts shall be drawn up tight and threads burred.

16. **Pin Connections.**—Pilot and driving nuts shall be used in driving pins. They will be furnished by the Company and shall be returned to the Company on completion of the work. Pin nuts shall be screwed up tight and threads burred.

17. **Deck.**—Where so specified, the ties, guard timbers, guard rails, fire decking, concrete decking, waterproofing, ballast, and deck planking, and the track rails and tie plates, shall be placed by the Contractor. The timber deck shall be placed in accordance with the Company's plans. If treated timber is used, the Company will deliver it properly framed to the Contractor. If untreated, it shall be framed by the Contractor. The ties shall be framed to give a full and even bearing on the girders and under the rails. The guard timbers shall be dapped and framed to a snug fit over the ties and fastened as shown on the plans. If necessary to do any framing or cutting of treated timber, the resulting surfaces shall be given a brush treatment with wood preservative, as directed by the Engineer. Where concrete decking is used, or waterproofing is required, the specifications therefore will be furnished by the Company.

18. **Misfits.**—Corrections of minor misfits and a reasonable amount of reaming and cutting of excess stock from rivets will be considered a legitimate part of the erection. Any error in shop work which prevents the proper assembling and fitting up of parts by the moderate use of drift pins or a moderate amount of reaming and slight chipping or cutting, shall immediately be reported to the Inspector, and his approval of the method of correction obtained. The correction shall be made in the presence of the Inspector, who will check the time and material. The Contractor shall render within thirty days an itemized bill for such work of correction for the approval of the Engineer.

19. **Painting.**—The heads of field rivets shall be given a coat of the shop paint by the Contractor. This painting shall not be done until the Inspector has examined the rivets and found them satisfactory. The tops of stringers and girders which are to carry ties shall be given one coat of field paint.

If the field painting is to be done by the Contractor, the specifications therefore will be furnished by the Company.

20. **Removal of Old Structure and Falsework.**—The Contractor shall dismantle the old structure and falsework and load the material on cars for shipment, or pile it neatly at a site immediately adjacent to the tracks, at an elevation convenient for future handling, as directed by the Engineer. When the old structure is of iron or steel and is to be used again, it shall be dismantled without unnecessary damage and the parts match-marked.

21. The Contractor shall remove the piling to the surface of the ground, and all debris and refuse resulting from his work, leaving the premises in good condition.

22. **Superintendence and Workmen.**—During the entire progress of the work the Contractor shall have a competent foreman or superintendent in personal charge of the work. Instructions

given to the foreman or superintendent shall be considered as given to the Contractor. All work shall be done by skilled, competent workmen.

23. **Interference With Traffic.**—The Contractor shall conduct his work in such a manner that the track, while in service, will be safe and clear for the passage of trains. Tracks shall be disturbed or removed for the prosecution of the work during such times only as allowed by the Company. While the Contractor is actively engaged in the erection, trains will be required to approach the bridge prepared to come to a stop before crossing and will proceed only on signal. During the time the Contractor operates his equipment on the tracks or has occasion to make the tracks unsafe for the operation of trains, his operations will be in charge of a conductor or pilot who will arrange and control the train movements.

24. **Company Equipment.**—When the agreement provides that the Company shall furnish equipment to the Contractor, such as flat cars, water cars, bunk cars, etc., the Contractor shall repair all damage to such equipment furnished for his use and return it in as good condition as when he received it.

25. **Work Train Service.**—When, under the agreement, work train or engine service is furnished the Contractor without charge, the Contractor shall state in his bid the number of days such service will be required. Any excess over the time specified in this bid shall be paid for by the Contractor at the Company's schedule of rates.

26. **Risk.**—The Contractor shall be responsible for loss of or damage to materials, for all damage to persons or property, and for casualties of every description caused by his operations during the progress of the work. Injuries or losses due to events beyond the control of the Contractor shall not be borne by him unless they occurred because of his dilatory methods in handling the work, extending the time beyond the time limit designated in the agreement.

27. **Inspection.**—The work shall be subject at all times to inspection by the Engineer.

28. **Laws and Permits.**—The Contractor shall comply with Federal, State and local laws, regulations and ordinances, and shall obtain at his own expense the necessary permits for his operations.

INSTRUCTIONS FOR THE INSPECTION OF BRIDGE ERECTION.*

(1) Study and observe the plans and specifications for steel construction. Study the masonry plans and check the masonry as built with the steel plans.

(2) Familiarize yourself with the local conditions affecting erection.

Make the acquaintance of the principal men engaged upon the work and of local residents whose interests may be affected thereby.

(3) Obtain and study carefully the time table and be well posted concerning the time when regular and extra trains are due and their relative importance. Acquaint yourself with all special traffic arrangements, made because of the work in hand.

(4) Secure full information concerning the conditions of the work in the bridge shop and the probable dates of shipment.

(5) Obtain reports of any uncompleted or erroneous work that must be attended to after arrival of the material in the field.

(6) Study the erection program in order to avoid delays and be able to recommend some other procedure in an emergency.

(7) Endeavor to have full preparations made before disturbing the track so that the erection may proceed rapidly and the period of such disturbance be made a minimum.

(8) Keep a record of the arrival of all materials. The contractor's record should be sufficient if available. Strive to anticipate any shortage of material and use all available facilities to hasten delivery of the needed parts.

(9) Study the progress of the work and determine whether it is likely to be completed in the time allotted. If not, endeavor to secure such additions to the force and equipment as will insure such completion.

(10) Make a daily record of the force employed and the distribution of labor, in a way that will assist in following clauses 9 and 23.

(11) Exercise a constant supervision of any temporary structure or falsework and make soundings if necessary with the purpose of discovering any evidence of failure or lack of safety and having it corrected before damage is done. Examine erection equipment with a view to its safety and adequacy.

(12) Be constantly on hand when work is in progress and note any damage to the metal, failure to conform to the specification or any especial difficulty in assembling.

(13) Make sure that each member of the structure is placed in its proper position. If match marks are used, examine them with care.

* Am. Ry. Eng. Assoc., Vol. 14, p. 90.

Endeavor to have the several members assembled in such order that no unsatisfactory makeshifts need be resorted to in getting some minor member in place.

(14) Prevent any abuse or rough usage of the material. Bending, straining and heavy pounding with sledges are included in such abuse.

(15) Watch carefully the use of fillers, washers and threaded members to see that they are neither omitted nor misused.

(16) Make certain that all parts of the structure are properly aligned and that the required camber exists before riveting. It is possible for a structure to be badly distorted although the rivet holes are well filled with the bolts.

(17) Watch the heating of rivets to insure against overheating and to make sure that scale is removed.

Examine and test carefully all field-driven rivets and have any that are loose or imperfect replaced.

Have cut out and replaced all rivets, whether shop-driven or field-driven, that may be loosened during erection and riveting.

Prevent injury to metal while removing rivets.

(18) Present to the contractor at once for his attention any violation of the specifications or contract, and secure a correction or refer the matter to the proper authorities as soon as possible.

(19) Keep informed concerning the use of Company material and work trains and assist in procuring such material and trains when needed, and preserve a record thereof.

(20) Secure a match-marking diagram of any old structure to be removed and see that each part of such structure is properly marked in accordance therewith. Make a record of the manner of cutting the old structure apart and report any damage to the members of the old structure. Indicate by sketches or otherwise such repairs or replacement as will be found necessary in re-erection.

(21) Secure photographic records of progress and the important features of the work wherever practicable.

(22) Make a record of flagging of trains, whether performed for the benefit of the Contractor or otherwise, delays to trains, personal injuries, and accidents of every kind.

(23) Make reports as directed, showing the progress of the work, the size of the force and the equipment in use.

Make a final report showing the cost of labor of erection per ton of material erected, the cost of labor per rivet in riveting, the cost of correcting errors in design and fabrication and commenting on the design and details; and give such other information as may be useful in planning similar work.

References.—For an investigation of the stresses in derricks and data on erection equipment, see Bland's " Handbook of Steel Erection," published by the McGraw-Hill Book Company, Inc.

CHAPTER XV.

ENGINEERING MATERIALS.

IRON AND STEEL.—The following definitions were adopted by the Committee on the Uniform Nomenclature of Iron and Steel of the International Association for Testing Materials, September, 1906.

Cast Iron.—Iron containing so much carbon or its equivalent that it is not malleable at any temperature. The committee recommends drawing the line between cast iron and steel at 2.20 per cent carbon.

Pig Iron.—Cast iron which has been cast into pigs direct from the blast furnace.

Bessemer Pig Iron.—Iron which contains so little phosphorus and sulphur that it can be used for conversion into steel by the original or acid Bessemer process (restricted to pig iron containing not more than 0.10 per cent of phosphorus).

Basic Pig Iron.—Pig iron containing so little silicon and sulphur that it is suited for easy conversion into steel by the basic open-hearth process (restricted to pig iron containing not more than 1.00 per cent of silicon).

Gray Pig Iron and Gray Cast Iron.—Pig iron and cast iron in the fracture of which the iron itself is nearly or quite concealed by graphite, so that the fracture has the gray color of graphite.

White Pig Iron and White Cast Iron.—Pig iron and cast iron in the fracture of which little or no graphite is visible, so that the fracture is silvery and white.

Malleable Castings.—Castings made from iron which when first made is in the condition of cast iron, and is made malleable by subsequent treatment without fusion.

Malleable Pig Iron.—An American trade name for the pig iron suitable for converting into malleable castings through the process of melting, treating when molten, casting in a brittle state, and then making malleable without remelting.

Wrought Iron.—Slag-bearing, malleable iron, which does not harden materially when suddenly cooled.

Steel.—Iron which is malleable at least in some one range of temperature and in addition is either (a) cast into an initially malleable mass; or, (b) is capable of hardening greatly by sudden cooling; or, (c) is both so cast and so capable of hardening.

Open-hearth Steel.—Steel made by the open-hearth process, irrespective of carbon content.

Bessemer Steel.—Steel made by the Bessemer process, irrespective of carbon content.

Blister Steel.—Steel made by carburizing wrought iron by heating it in contact with carbonaceous matter.

Crucible Steel.—Steel made by the crucible process, irrespective of carbon content.

Steel Castings.—Unforged and unrolled castings made of Bessemer, open-hearth, crucible or any other steel.

Alloy Steels.—Steels which owe their properties chiefly to the presence of an element other than carbon.

Classification of Iron and Steel.—The limits of carbon, the specific gravity and properties of iron and steel are as follows:

	Per cent of Carbon.	Specific Gravity.	Properties.
Cast Iron	5 to 1.50	7.2	Not malleable, not temperable
Steel	1.50 to 0.10	7.8	Malleable and temperable
Wrought Iron	0.30 to 0.05	7.7	Malleable, not temperable

It will be seen that the percentage of carbon alone is not sufficient to distinguish between steel and wrought iron. The softer grades of steel resemble wrought iron. Very mild open-hearth steel is often sold under the trade name of "Ingot Iron," and is reputed to have many advantages over structural steel, most of which properties it does not possess among which is the ability to resist corrosion.

CAST IRON.—The product of the blast furnace, where the iron ore is reduced in the presence of a flux, is called pig iron. The term cast iron is commonly applied to pig iron after it has been again melted and cast into finished form. Cast iron contains carbon, silicon, sulphur, phosphorus, and manganese in addition to pure iron, and occasionally very small quantities of other elements. The amount of carbon depends largely upon the presence of other elements.

Carbon.—The percentage of carbon ordinarily varies between 1½ and 4 per cent, but in the presence of manganese the carbon may be much higher. Carbon may occur in the form of combined carbon, giving a white brittle cast iron, or in the form of graphite, giving a gray cast iron, which is the form used in structural castings. The proper amount of carbon in cast iron depends upon the amount of other impurities and upon the use that is to be made of the finished product.

Silicon.—The carbon is controlled by varying the amount of silicon and sulphur. Silicon acts as a precipitant of carbon, changing it from the combined form to the graphite form. The silicon in gray cast iron is usually between ¾ and 3 per cent.

Sulphur.—Sulphur has the opposite effect of silicon and its presence is considered objectionable. Sulphur produces "red-shortness" (brittleness when the iron is heated). The amount of sulphur in gray-iron castings should not exceed 0.12 per cent.

Manganese.—Manganese and sulphur both tend to increase the amount of combined carbon, but they tend to neutralize each other. Manganese gives closeness of grain and prevents the absorption of sulphur on remelting. The amount of manganese in gray-iron castings is usually less than ½ per cent; more than 2 per cent makes cast iron brittle.

Phosphorus.—Phosphorus increases the fusibility and fluidity of cast iron but at the same time makes it brittle. A high phosphorus content is necessary in cast iron for light ornamental castings where strength is not required. The phosphorus in gray-iron castings varies from ½ to 1½ per cent.

Malleable Castings.—Small thin castings made of white cast iron may be decarbonized by heating the castings in annealing pots containing hematite ore or forge iron scale. The castings are kept at a cherry red heat for three to four days, and are then allowed to cool slowly. The metal in malleable castings should not exceed ¼ in. in thickness in small castings, nor ½ in. in large castings, and should be of uniform thickness.

Strength of Cast Iron.—The strengths of gray-iron castings are given in Table I and in the Specifications for Gray-iron Castings of the American Society for Testing Materials.

STANDARD SPECIFICATIONS FOR GRAY-IRON CASTINGS OF THE AMERICAN SOCIETY FOR TESTING MATERIALS.

ADOPTED SEPTEMBER 1, 1905.*

1. **Process of Manufacture.** All gray castings are understood to be made by the cupola process, unless furnace iron is specified.

2. **Chemical Properties.** The sulphur contents to be as follows:

Light castings	not over 0.08 per cent
Medium castings	" 0.10 "
Heavy castings	" 0.12 "

3. **Classification.** In dividing castings into light, medium and heavy classes, the following standards have been adopted:

Castings having any section less than ½ in. thick shall be known as *light castings.*
Castings in which no section is less than 2 in. thick shall be known as *heavy castings.*
Medium castings are those not included in the above classification.

4. **Physical Properties.** *Transverse Test.* The minimum breaking strength of the "Arbitration Bar" under transverse load shall be not under:

Light castings	2,500 lb.
Medium castings	2,900 "
Heavy castings	3,300 "

In no case shall the deflection be under 0.10 in.

* Details of making tests revised in 1918. Physical and chemical requirements were not changed.

Tensile Test. Where specified, this shall not run less than:

Light castings	18,000 lb. per sq. in.
Medium castings	21,000 " " "
Heavy castings	24,000 " " "

5. **Arbitration Bar.** The quality of the iron going into castings under specification shall be determined by means of the " Arbitration Bar." This is a bar $1\frac{1}{4}$ in. in diameter and 15 in. long. It shall be prepared as stated further on and tested transversely. The tensile test is not recommended, but in case it is called for, the bar as shown in Fig. 1, and turned up from any of the broken pieces of the transverse test shall be used. The expense of the tensile test shall fall on the purchaser.

6. **Number of Test Bars.** Two sets of two bars shall be cast from each heat, one set from the first and the other set from the last iron going into the castings. Where the heat exceeds twenty tons, an additional set of two bars shall be cast for each twenty tons or fraction thereof above this amount. In case of a change of mixture during the heat, one set of two bars shall also be cast for every mixture other than the regular one. Each set of two bars is to go into a single mold. The bars shall not be rumbled or otherwise treated, being simply brushed off before testing.

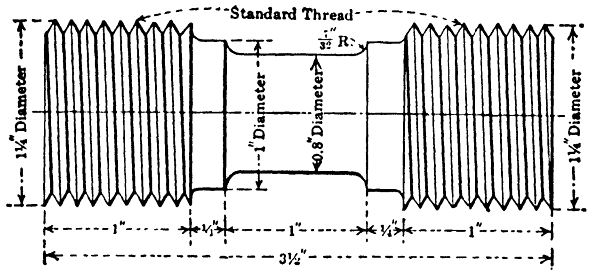


FIG. 1.—ARBITRATION TEST BAR. TENSILE TEST PIECE

7. **Method of Testing.** The transverse test shall be made on all the bars cast, with supports 12 in. apart, load applied at the middle, and the deflection at rupture noted. One bar of every two of each set made must fulfil the requirements to permit acceptance of the castings represented.

8. **Mold for Test Bar.** The mold for the bars is shown in Fig. 2. The bottom of the bar is $\frac{1}{16}$ in. smaller in diameter than the top, to allow for draft and for the strain of pouring. The pattern shall not be rapped before withdrawing. The flask is to be rammed up with green molding sand, a little damper than usual, well mixed and put through a No. 8 sieve, with a mixture of one to twelve bituminous facing. The mold shall be rammed evenly and fairly hard, thoroughly dried and not cast until it is cold. The test bar shall not be removed from the mold until cold enough to be handled.

9. **Speed of Testing.** The rate of application of the load shall be from 20 to 40 seconds for a deflection of 0.10 in.

10. **Samples for Analysis.** Borings from the broken pieces of the " Arbitration Bar " shall be used for the sulphur determinations. One determination for each mold made shall be required. In case of dispute, the standards of the American Foundrymen's Association shall be used for comparison.

11. **Finish.** Castings shall be true to pattern, free from cracks, flaws and excessive shrinkage. In other respects they shall conform to whatever points may be specially agreed upon.

12. **Inspection.** The inspector shall have reasonable facilities afforded him by the manufacturer to satisfy him that the finished material is furnished in accordance with these specifications. All tests and inspections shall, as far as possible, be made at the place of manufacture prior to shipment.

WROUGHT IRON.—Wrought iron is made in a reverberatory furnace from pig iron or from molten metal taken directly from the blast furnace. The hearth of the reverberatory furnace is fettled with high grade iron ore or mill scale, which acts as an oxidizing agent for reducing the impurities. The puddling process may be divided into four stages: First or melting down stage, occupying about 30 minutes, during which the silicon and manganese are oxidized and a consider-

able part of the phosphorus is oxidized; all oxidized products unite with the slag. Second or clearing stage, occupying about 10 minutes, during which the remainder of the silicon and manganese, and more of the phosphorus are oxidized and removed from the pig iron. Third or boiling stage, occupying about 30 minutes, in which nearly all the carbon is removed and most of the remaining phosphorus is removed. Last or balling stage, occupying about 20 minutes, in which the metal is gathered by the puddler into balls weighing about 75 to 100 lb.

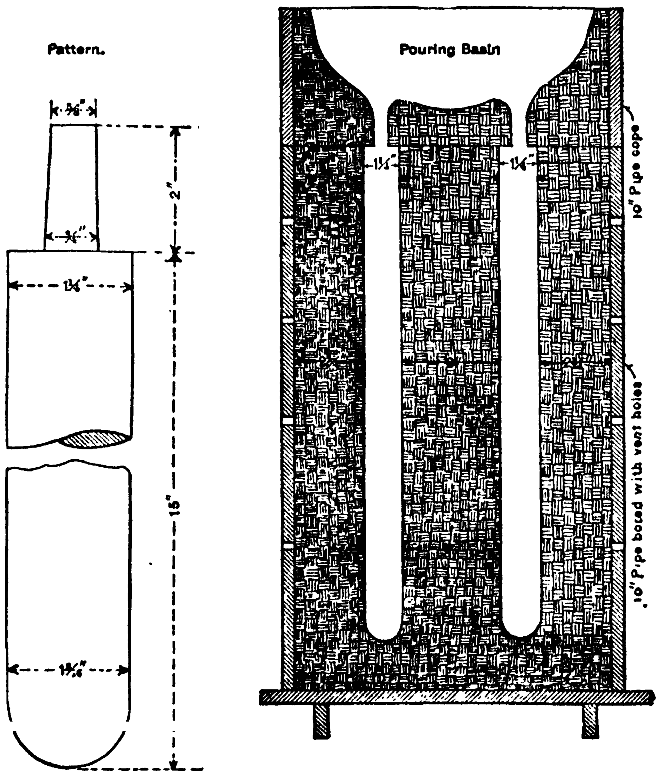


FIG. 2.—MOLD FOR ARBITRATION TEST BAR.

The puddled balls of iron and slag are hammered or are run through rolls to squeeze the slag from the balls, and the resulting bars are called muck bars. The muck bar is again reheated and rerolled and the resulting product is commercial merchant bar.

Wrought iron when broken in tension shows a fractured section irregular and fibrous. The strength of wrought iron varies with the chemical composition, the mechanical work and heat treatment it has received. The strength of wrought iron is given in Table I, and the specifications for wrought-iron bars and plates as adopted by the American Society for Testing Materials are as follows:

STANDARD SPECIFICATIONS FOR REFINED WROUGHT-IRON BARS

OF THE

AMERICAN SOCIETY FOR TESTING MATERIALS.

ADOPTED 1912; REVISED 1918.

I. MANUFACTURE.

1. **Process.** Refined wrought-iron bars shall be made wholly from puddled iron, and may consist either of new muck-bar iron or a mixture of muck-bar iron and scrap, but shall be free from any admixture of steel.

II. PHYSICAL PROPERTIES AND TESTS.

2. **Tension Tests.** (*a*) The iron shall conform to the following minimum requirements as to tensile properties:

Tensile strength, lb. per sq. in. (See Sections 3 and 4.)	48,000
Yield point, lb. per sq. in.	25,000
Elongation in 8 in., per cent (See Section 5.)	22

(*b*) The yield point shall be determined by the drop of the beam of the testing machine. The speed of the cross-head of the machine shall not exceed ¾ in. per minute.

3. **Permissible Variations in Tensile Strength.** Twenty per cent of the test specimens representing one size may show tensile strengths 1000 lb. per sq. in. under or 5000 lb. per sq. in. over that specified in Section 2; but no specimen shall show a tensile strength under 45,000 lb. per sq. in.

4. **Modifications in Tensile Strength.** For material over 4 sq. in. in sectional area, a reduction of 500 lb. per sq. in. from the tensile strength specified in Section 2 will be permitted for each additional 2 sq. in., and a proportionate amount of reduction for fractional parts thereof; provided that the tensile strength shall not be less than 45,000 lb. per sq. in.

5. **Permissible Variations in Elongation.** Twenty per cent of the test specimens representing one size may show the following percentages of elongation in 8 in.:

ROUND BARS.

½ in. or over, tested as rolled	20 per cent
Under ½ in., " " "	16 "
Reduced by machining	18 "

FLAT BARS.

⅜ in. or over, tested as rolled	18 per cent
Under ⅜ in., " " "	16 "
Reduced by machining	16 "

6. **Bend Tests.** (*a*) *Cold-bend Tests.*—Cold-bend tests will be made only on bars having a nominal area of 4 sq. in. or under, in which case the test specimen shall bend cold through 180 deg. without fracture on the outside of the bent portion, around a pin the diameter of which is equal to twice the diameter or thickness of the specimen.

(*b*) *Hot-bend Tests.*—The test specimen, when heated to a temperature between 1700° and 1800° F., shall bend through 180 deg. without fracture on the outside of the bent portion, as follows: For round bars under 2 sq. in. in section, flat on itself; for round bars 2 sq. in. or over in section and for all flat bars, around a pin the diameter of which is equal to the diameter or thickness of the specimen.

(*c*) *Nick-bend Tests.*—The test specimen, when nicked 25 per cent around for round bars, and along one side for flat bars, with a tool having a 60-deg. cutting edge, to a depth of not less than 8 nor more than 16 per cent of the diameter or thickness of the specimen, and broken, shall not show more than 10 per cent of the fractured surface to be crystalline.

(*d*) Bend tests may be made by pressure or by blows.

7. **Etch Tests.*** The cross-section of the test specimen shall be ground or polished, and etched for a sufficient period to develop the structure. This test shall show the material to be free from steel.

*A solution of two parts water, one part concentrated hydrochloric acid, and one part concentrated sulphuric acid is recommended for the etch test.

8. **Test Specimens.** (*a*) Tension and bend test specimens shall be of the full section of material as rolled, if possible. Otherwise, the specimens shall be machined from the material as rolled; the axis of the specimen shall be located at any point one-half the distance from the center to the surface of round bars, or from the center to the edge of flat bars, and shall be parallel to the axis of the bar.

(*b*) Etch test specimens shall be of the full section of material as rolled.

9. **Number of Tests.** (*a*) All bars of one size shall be piled separately. One bar from each 100 or fraction thereof will be selected at random and tested as specified.

(*b*) If any test specimen from the bar originally selected to represent a lot of material, contains surface defects not visible before testing but visible after testing, or if a tension test specimen breaks outside the middle third of the gage length, one retest from a different bar will be allowed.

III. PERMISSIBLE VARIATIONS IN GAGE.

10. **Permissible Variations.** (*a*) Round bars shall conform to the standard limit gages adopted by the Master Car Builders' Association in 1883, revised in 1911.

(*b*) The width or thickness of flat bars shall not vary more than 2 per cent from that specified.

IV. FINISH.

11. **Finish.** The bars shall be smoothly rolled and free from slivers, depressions, seams, crop ends, and evidences of being burnt.

V. INSPECTION AND REJECTION.

12. **Inspection.** (*a*) The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. Tests and inspection at the place of manufacture shall be made prior to shipment.

(*b*) The purchaser may make the tests to govern the acceptance or rejection of material in his own laboratory or elsewhere. Such tests, however, shall be made at the expense of the purchaser.

13. **Rejection.** All bars of one size will be rejected if the test specimens representing that size do not conform to the requirements specified.

STANDARD SPECIFICATIONS FOR WROUGHT-IRON PLATES

OF THE

AMERICAN SOCIETY FOR TESTING MATERIALS.

ADOPTED AUGUST 25, 1913; REVISED 1918.

1. **Classes.** These specifications cover two classes of wrought-iron plates, namely:
   Class A, as defined in Section 2 (*b*);
   Class B, as defined in Section 2 (*c*).

I. MANUFACTURE.

2. **Process.** (*a*) All plates shall be rolled from piles entirely free from any admixture of steel.

(*b*) Piles for Class A plates shall be made from puddle bars made wholly from pig iron and such scrap as emanates from rolling the plates.

(*c*) Piles for Class B plates shall be made from puddle bars made wholly from pig iron or from a mixture of pig iron and cast-iron scrap, together with wrought-iron scrap.

II. PHYSICAL PROPERTIES AND TESTS.

3. **Tension Tests.** (*a*) The plates shall conform to the following minimum requirements as to tensile properties:

(*b*) The yield point shall be determined by the drop of the beam of the testing machine. The speed of the cross-head of the machine shall not exceed $\frac{3}{4}$ in. per minute.

Properties Considered.	Class A.		Class B.	
	6 In. to 24 In., Incl., in Width.	Over 24 In. to 90 In., Incl., in Width.	6 In. to 24 In., Incl., in Width.	Over 24 In. to 90 In., Incl., in Width.
Tensile strength, lb. per sq. in.	49,000	48,000	48,000	47,000
Elastic limit, lb. per sq. in.	26,000	26,000	26,000	26,000
Elongation in 8 in., per cent	16	12	14	10

4. **Modifications in Elongation.** For plates under $\frac{7}{16}$ in. in thickness, a deduction of 1 from the percentages of elongation specified in Section 3 shall be made for each decrease of $\frac{1}{16}$ in. in thickness below $\frac{7}{16}$ in.

5. **Bend Tests.** (*a*) *Cold-bend Tests.*—The test specimen shall bend cold through 90 deg. without fracture on the outside of the bent portion, as follows: For Class A plates, around a pin the diameter of which is equal to $1\frac{1}{2}$ times the thickness of the specimen; and for Class B plates, around a pin the diameter of which is equal to 3 times the thickness of the specimen.

(*b*) *Nick-bend Tests.*—The test specimen, when nicked on one side and broken, shall show for Class A plates a wholly fibrous fracture, and for Class B plates, not more than 10 per cent of the fractured surface to be crystalline.

6. **Test Specimens.** Tension and bend test specimens shall be taken from the finished plates and shall be of the full thickness of plates as rolled. The longitudinal axis of the specimen shall be parallel to the direction in which the plates are rolled.

7. **Number of Tests.** (*a*) One tension, one cold-bend and one nick-bend test shall be made for each variation in thickness of $\frac{1}{8}$ in. and not less than one test for every ten plates as rolled.

(*b*) If any test specimen fails to conform to the requirements specified through an apparent local defect, a retest shall be taken; and should the retest fail, the plates represented by such test shall be rejected.

III. FINISH.

8. **Finish.** The plates shall be straight, smooth and free from cinder spots and holes, and free from injurious flaws, buckles, blisters, seams and laminations.

IV. MARKING.

9. **Marking.** The plates shall be stamped or otherwise marked as designated by the purchaser.

V. INSPECTION AND REJECTION.

10. **Inspection.** (*a*) The inspector representing the purchaser shall have free entry at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the plates ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the plates are being furnished in accordance with these specifications. Tests and inspection at the place of manufacture shall be made prior to shipment.

(*b*) The purchaser may make the tests to govern the acceptance or rejection of plates at his own laboratory or elsewhere. Such tests, however, shall be made at the expense of the purchaser.

(*c*) All tests and inspection shall be so conducted as not to interfere unnecessarily with the operation of the works.

11. **Rejection.** Unless otherwise specified, any rejection based on tests made in accordance with Section 10 (*b*) shall be reported within five working days from the receipt of samples.

12. **Rehearing.** Samples tested in accordance with Section 10 (*b*), which represent rejected plates, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

STEEL.—The three principal methods for the manufacture of steel are (1) the crucible process, (2) the Bessemer process, and (3) the open-hearth process. The crucible process is used for making tool steel. The Bessemer process is used for making structural steel, but on account of its requiring a high grade ore for a satisfactory steel, and the difficulty of control, it is now practically replaced by the open-hearth process. The following description of the methods of manufacture of steel is taken from Kent's "Mechanical Engineer's Pocket-Book," page 451, 8th Edition, 1910.

The Manufacture of Steel.—Cast steel is a malleable alloy of iron, cast from a fluid mass. It is distinguished from cast iron, which is not malleable, by being much lower in carbon, and from wrought iron, which is welded from a pasty mass, by being free from intermingled slag. Blister steel is a highly carbonized wrought iron, made by the "cementation" process, which consists in keeping wrought-iron bars at a red heat for some days in contact with charcoal. Not over 2 per cent of C is usually absorbed. The surface of the iron is covered with small blisters, supposedly due to the action of carbon on slag. Other wrought steels were formerly made by direct processes from iron ore, and by the puddling process from wrought iron, but these steels are now replaced by cast steels. Blister steel is, however, still used as a raw material in the manufacture of crucible steel. Case-hardening is a process of surface cementation.

Crucible Steel is commonly made in pots or crucibles holding about 80 pounds of metal. The raw material may be steel scrap; blister steel bars; wrought iron with charcoal; cast iron with wrought iron or with iron ore; or any mixture that will produce a metal having the desired chemical constitution. Manganese in some form is usually added to prevent oxidation of the iron. Some silicon is usually absorbed from the crucible, and carbon also if the crucible is made of graphite and clay. The crucible being covered, the steel is not affected by the oxygen or sulphur in the flame The quality of crucible steel depends on the freedom from objectionable elements, such as phosphorus, in the mixture, on the complete removal of oxide, slag and blowholes by "dead-melting" or "killing" before pouring, and on the kind and quantity of different elements which are added in the mixture, or after melting, to give particular qualities to the steel, such as carbon, manganese, chromium, tungsten and vanadium.

Bessemer Steel is made by blowing air through a bath of melted pig iron. The oxygen of the air first burns away the silicon, then the carbon, and before the carbon is entirely burned away, begins to burn the iron. Spiegeleisen or ferro-manganese is then added to deoxidize the metal and to give it the amount of carbon desired in the finished steel. In the ordinary or "acid" Bessemer process the lining of the converter is a silicious material, which has no effect on phosphorus, and all the phosphorus in the pig iron remains in the steel. In the "basic" or Thomas and Gilchrist process the lining is of magnesian limestone, and limestone additions are made to the bath, so as to keep the slag basic; and the phosphorus enters the slag. By this process ores that were formerly unsuited to the manufacture of steel have been made available.

Open-hearth Steel.—Any mixture that may be used for making steel in a crucible may also be melted on the open hearth of a Siemens regenerative furnace, and may be desiliconized and decarbonized by the action of the flame and by additions of iron ore, deoxidized by the addition of spiegeleisen or ferro-manganese, and recarbonized by the same additions or by pig iron. In the most common form of the process pig iron and scrap steel are melted together on the hearth, and after the manganese has been added to the bath it is tapped into the ladle. In the Talbot process a large bath of melted material is kept in the furnace, melted pig iron, taken from a blast furnace, is added to it, and iron ore is added which contributes its iron to the melted metal while its oxygen decarbonizes the pig iron. When the decarbonization has proceeded far enough, ferro-manganese is added to destroy iron oxide, and a portion of the metal is tapped out, leaving the remainder to receive another charge of pig iron, and thus the process is continued indefinitely. In the Duplex process melted cast iron is desiliconized in a Bessemer converter, and then run into an open hearth, where the steel-making operation is finished.

The open-hearth process, like the Bessemer, may be either acid or basic, according to the character of the lining. The basic process is a dephosphorizing one, and is the one most generally available, as it can use pig irons that are either low or high in phosphorus.

Strength of Steel.—The properties most desired in steel are strength and ductility. Pure iron has a tensile strength of about 40,000 lb. per sq. in. and is very ductile. This strength is usually increased by the impurities found in steel.

Carbon is the important impurity as it gives strength with the least decrease in ductility. Campbell states that each 0.01 per cent of carbon will increase the strength of acid open-hearth steel by 1000 lb. per sq. in., and of basic open-hearth steel by 770 lb. per sq. in. The maximum tensile strength of steel is reached with 0.9 to 1.0 per cent of carbon.

Silicon has little effect on the strength of rolled steel, but in castings 0.3 to 0.4 per cent of silicon increases the tensile strength of steel castings and produces soundness.

Sulphur has little effect on the strength of open-hearth steel, but it produces "red-shortness," and produces checks and cracks during the rolling or during the cooling of castings.

Phosphorus increases the ***static*** strength of steel about 1000 lb. for each 0.01 per cent of phosphorus. The increase in strength is obtained at a great loss in ductility and produces a steel that is brittle and unreliable.

Manganese when above 0.3 to 0.4 per cent increases the tensile strength of steel. The increase in strength above 0.4 per cent is about 300 lb. per sq. in. for acid open-hearth and 130 lb. per sq. in. for basic open-hearth steel for each additional 0.01 per cent of manganese.

From the above discussion it will be seen that if certain physical characteristics are required in a steel the manufacturer must be left free to vary part of the impurities. For example if a high grade structural steel with an ultimate tensile strength of 60,000 lb. per sq. in. is desired, the phosphorus and sulphur may be limited in addition to the prescribed physical limits if the carbon is left open.

Formulas for Tensile Strength.—Campbell gives the following formulas for the strength of acid and basic open-hearth steels:

For acid steel, Ultimate strength = 40,000 + 1000 C + 1000 P + X.Mn + R.

For basic steel, Ultimate strength = 41,500 + 770 C + 1000 P + X.Mn + R.

In these formulas, C = 0.01 per cent carbon, P = 0.01 phosphorus, Mn = 0.01 per cent manganese above 0.4 per cent for acid and above 0.3 per cent for basic steel, and R is a variable depending upon the heat treatment of the steel. The coefficient of Mn, X, varies as follows: For acid steel, for 0.10 per cent carbon, X = 80, and for 0.60 per cent carbon, X = 480 and proportional for intermediate values; while for basic steel, for 0.05 per cent carbon, X = 110, and for 0.40 per cent carbon, X = 250 and proportional for intermediate values.

Special Steels.—The following special steels have been used. *Nickel* is used as an alloy for structural and other kinds of steel, the specifications for structural nickel steel of the American Society for Testing Materials require that there be not less than $3\frac{1}{4}$ per cent of nickel. *Chrome steel*—carbon steel with about 0.5 per cent chromium—was used in the Eads bridge in 1871. Chromium is now used in combination with nickel, making *Chromium-nickel* steel; with vanadium, making *Chromium-vanadium* steel, and with both nickel and vanadium, making *Chromium-nickel-vanadium* steel. *Copper* steels are those having from 1 to 4 per cent of copper, carbon being less than 1 per cent. *Manganese* steel with from 6 to 12 per cent manganese is very tough and malleable.

Specifications for Structural Steel.—The allowable stresses for structural steel are given in Table I and in the specifications of the American Society for Testing Materials which follow.

Allowable Stresses in Steel and Iron.—The allowable stresses for steel frame mill buildings are given in the "Specifications for Steel Frame Buildings," in Chapter I. The allowable stresses for steel office buildings are given in the "Specifications for Steel Office Buildings," in Chapter II. The allowable stresses for steel highway bridges are given in the "Specifications for Steel Highway Bridges," in Chapter III. The allowable stresses for steel railway bridges are given in the "Specifications for Steel Railway Bridges," in Chapter IV. The allowable stresses in steel bins are given in Chapter VIII. The allowable stresses for steel grain bins are given in Chapter IX. The allowable stresses in steel head frames and coal tipples are given in the "Specifications for Steel Head Frames and Coal Tipples, Washers and Breakers" in Chapter X. The allowable stresses in steel stand-pipes and elevated tanks are given in the "Specifications for Elevated Steel Tanks on Towers and for Stand-Pipes," in Chapter XI. The allowable stresses for the steel and cast iron details in timber bridges are the same as for steel railway bridges given in Chapter IV. The allowable stresses in steel reinforcement are given at end of this chapter.

Nickel Steel.—In a paper entitled "Nickel Steel for Bridges" by Mr. J. A. L. Waddell, in Trans. Am. Soc. C. E., Vol. 63, June 1909, the allowable unit stress in lb. per sq. in. for carbon steel is given as $P = 18{,}000 - 70l/r$, and for nickel steel as $P = 30{,}000 - 120l/r$, where l is the length and r is the corresponding radius of gyration, both in inches.

For a detail discussion of the manufacture of structural steel see "The Making, Shaping and Treating of Steel," by J. M. Camp and C. B. Francis, published by Carnegie Steel Company.

TABLE I.

STRENGTH PROPERTIES OF STRUCTURAL STEEL AND IRON—AMERICAN SOCIETY FOR TESTING MATERIALS, YEAR BOOK, 1923.

Metal.	Tensile Strength, Lb. Sq. In.		Minimum Elongation, Per Cent.		Reduction of Area, Per Cent.
	Ultimate.	Elastic Limit.	In 8 In.	In 2 In.	
BRIDGES					
Structural Steel	55,000–65,000	½ ultimate	$\frac{1,500,000}{\text{ultimate}}$	22	
Rivet Steel	46,000–56,000	½ ultimate	$\frac{1,5000,00}{\text{ultimate}}$		
BUILDINGS					
Structural Steel	55,000–65,000	½ ultimate	$\frac{1,400,000}{\text{ultimate}}$	22	
Rivet Steel	46,000–56,000	½ ultimate	$\frac{1,400,000}{\text{ultimate}}$		
SHIPS					
Structural Steel	58,000–68,000	½ ultimate	$\frac{1,500,000}{\text{ultimate}}$		
Rivet Steel	55,000–65,000	½ ultimate	$\frac{1,500,000}{\text{ultimate}}$		
BOILER AND RIVET STEEL					
Flange Steel	55,000–65,000	½ ultimate	$\frac{1,500,000}{\text{ultimate}}$		
Firebox Steel	52,000–62,000	½ ultimate	$\frac{1,500,000}{\text{ultimate}}$		
Boiler Rivet Steel	45,000–55,000	½ ultimate	$\frac{1,500,000}{\text{ultimate}}$ (not greater than 30)		
STRUCTURAL NICKEL STEEL					
Plates, Shapes and Bars	85,000–100,000	50,000	$\frac{1,500,000}{\text{ultimate}}$		25
Eye-bars and rollers (unannealed)	95,000–110,000	55,000	$\frac{1,500,000}{\text{ultimate}}$	16	25
Eye-bars and Pins (annealed)	90,000–105,000	52,000	20	20	35
Rivet Steel	70,000–80,000	45,000	$\frac{1,500,000}{\text{ultimate}}$		40
BILLET-STEEL REINFORCEMENT BARS					
Plain: Structural	55,000–70,000	33,000	$\frac{1,400,000}{\text{ultimate}}$		
Plain: Hard	80,000 min.	50,000	$\frac{1,200,000}{\text{ultimate}}$		
Deformed: Structural	55,000–70,000	33,000	$\frac{1,250,000}{\text{ultimate}}$		
Deformed: Hard	80,000 min.	50,000	$\frac{1,000,000}{\text{ultimate}}$		
Cold Twisted	recorded only	55,000	5		
RAIL-STEEL REINFORCEMENT BARS					
Plain	80,000	50,000	$\frac{1,200,000}{\text{ultimate}}$		
Deformed and Hot-twisted	80,000	50,000	$\frac{1,000,000}{\text{ultimate}}$		
WROUGHT IRON					
Refined Bars	48,000	25,000	22		
Plates	47,000–49,000	26,000	10 to 16		
STEEL CASTINGS					
Hard	80,000	36,000		15	20
Medium	70,000	31,500		18	25
Soft	60,000	27,000		22	30
GRAY IRON CASTINGS					
Light Castings	18,000				
Medium Castings	21,000				
Heavy Castings	24,000				
MALLEABLE CASTINGS	40,000			2½	

STANDARD SPECIFICATIONS FOR STRUCTURAL STEEL FOR BUILDINGS

OF THE

AMERICAN SOCIETY FOR TESTING MATERIALS.

ADOPTED AUGUST 25, 1913; REVISED 1921.

I. MANUFACTURE.

1. **Process.** (*a*) Structural steel, except as noted in Paragraph (*b*), may be made by the Bessemer or the open-hearth process.

(*b*) Rivet steel, and steel for plates or angles over ¾ in. in thickness which are to be punched, shall be made by the open-hearth process.

II. CHEMICAL PROPERTIES AND TESTS.

2. **Chemical Composition.** The steel shall conform to the following requirements as to chemical composition:

		Structural Steel.	Rivet Steel.
Phosphorus	Bessemer......	not over 0.10 per cent	......
	Open-hearth....	" " 0.06 "	not over 0.06 per cent
Sulphur..................		" "	" " 0.045 "

3. **Ladle Analyses.** An analysis to determine the percentages of carbon, manganese, phosphorus and sulphur shall be made by the manufacturer from a test ingot taken during the pouring of each melt, a copy of which shall be given to the purchaser or his representative. This analysis shall conform to the requirements specified in Section 2.

4. **Check Analyses.** Analyses may be made by the purchaser from finished material representing each melt, in which case an excess of 25 per cent above the requirements specified in Section 2 shall be allowed.

III. PHYSICAL PROPERTIES AND TESTS.

5. **Tension Tests.** (*a*) The material shall conform to the following requirements as to tensile properties:

Properties Considered.	Structural Steel.	Rivet Steel.
Tensile strength, lb. per sq. in............	55,000–65,000	46,000–56,000
Yield point, min., " "	0.5 tens. str.	0.5 tens. str.
Elongation in 8 in., min., per cent.........	$\frac{1,400,000}{\text{Tens. str.}}$ [1]	$\frac{1,400,000}{\text{Tens. str.}}$
Elongation in 2 in. " "	22	....

[1] See Section 6.

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

6. **Modifications in Elongation.** (*a*) For structural steel over ¾ in. in thickness, a deduction from the percentage of elongation in 8 in. specified in Section 5 (*a*) of 0.25 per cent shall be made for each increase of 1/32 in. of the specified thickness above ¾ in., to a minimum of 18 per cent.

(*b*) For structural steel under 5/16 in. in thickness, a deduction from the percentage of elongation in 8 in. specified in Section 5 (*a*) of 1.25 per cent shall be made for each decrease of 1/32 in. of the specified thickness below 5/16 in.

7. **Bend Tests.** (*a*) The test specimen for plates, shapes and bars, except as specified in Paragraphs (*b*) and (*c*), shall bend cold through 180 deg. without cracking on the outside of the bent portion, as follows: For material ¾ in. or under in thickness, flat on itself; for material over ¾ in. to and including 1¼ in. in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material over 1¼ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*) The 1 by ½-in. test specimen for pins, rollers and other bars, when prepared as specified in Section 8, shall withstand being bent cold through 180 deg. around a pin 1 in. in diameter.

(*c*) The test specimen for rivet steel shall bend cold through 180 deg. flat on itself without cracking on the outside of the bent portion.

8. **Test Specimens.** (*a*) Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed; in which case the test specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*b*) Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*d*), (*e*) and (*f*) shall be of the full thickness or diameter of material as rolled.

(*c*) Test specimens for plates, shapes and flats may be machined to the form and dimensions shown in Fig. 1, or with both edges parallel.

(*d*) Test specimens for plates over 1½ in. in thickness may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.

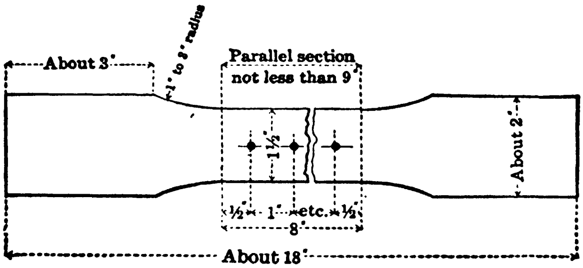
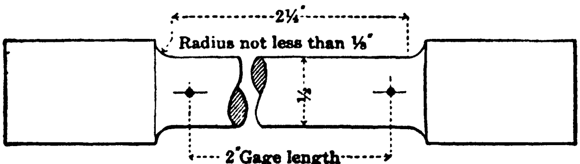


FIG. 1.

(*e*) Test specimens for bars over 1½ in. in thickness or diameter may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.; or tension test specimens may conform to the dimensions shown in Fig. 2, in which case the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend test specimens may be 1 by ½ in. in section.

(*f*) Tension test specimens for pins and rollers shall conform to the dimensions shown in Fig. 2. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend test specimens shall be 1 by ½ in. in section.

(*g*) The tension test specimen shown in Fig. 2 and the 1 by ½-in. bend test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars over 1½ in. in thickness or diameter, midway between the center and surface.



NOTE:—The gage length, parallel portions and fillets shall be as shown, but the ends may be of any form which will fit the holders of the testing machine.

FIG. 2.

(*h*) The machined sides of rectangular bend test specimens may have the corners rounded to a radius not over $\frac{1}{16}$ in.

(*i*) Test specimens for rivet bars which have been cold drawn shall be normalized before testing.

9 **Number of Tests.** (*a*) One tension and one bend test shall be made from each melt; except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 5 (*a*) and any part of the fracture is more than ¾ in. from the center of the gage length

Table I.

Permissible Variations of Plates Ordered to Weight.

Ordered Weight, Lb. per Sq. Ft.	Permissible Variations in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Ordered Weights.																		Ordered Weight, Lb. per Sq. Ft.
	Under 48 in.		48 to 60 in., excl.		60 to 72 in., excl.		72 to 84 in., excl.		84 to 96 in., excl.		96 to 108 in., excl.		108 to 120 in., excl.		120 to 132 in., excl.		132 in. or over.		
Under 5....	5	3	5.5	3	6	3	7	3	...	...	...	...	...	...	...	...	...	...	Under 5
5 to 7.5 exclusive....	4.5	3	5	3	5.5	3	6	3	...	...	...	...	...	...	...	...	...	...	5 to 7.5 exclusive
7.5 to 10 exclusive....	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	...	...	...	...	7.5 to 10 exclusive
10 to 12.5 exclusive....	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	9	3	10 to 12.5 exclusive
12.5 to 15 exclusive....	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	12.5 to 15 exclusive
15 to 17.5 exclusive....	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	15 to 17.5 exclusive
17.5 to 20 exclusive....	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	17.5 to 20 exclusive
20 to 25 exclusive....	2	2	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	20 to 25 exclusive
25 to 30 exclusive....	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	5	3	25 to 30 exclusive
30 to 40 exclusive....	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	30 to 40 exclusive
40 or over	2	2	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	40 or over

Note.—The weight per square foot of individual plates shall not vary from the ordered weight by more than $1\frac{1}{3}$ times the amount given in this table.

Table II.

Permissible Overweights of Plates Ordered to Thickness.

Ordered Thickness, in.	Permissible Excess in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Nominal Weights.									Ordered Thickness, in.
	Under 48 in.	48 to 60 in., excl.	60 to 72 in., excl.	72 to 84 in., excl	84 to 96 in., excl.	96 to 108 in., excl.	108 to 120 in., excl	120 to 132 in., excl.	132 in or over.	
Under $\frac{1}{8}$	9	10	12	14	..	..	..	..	..	Under $\frac{1}{8}$
$\frac{1}{8}$ to $\frac{3}{16}$ excl.	8	9	10	12	..	..	..	..	..	$\frac{1}{8}$ to $\frac{3}{16}$ excl.
$\frac{3}{16}$ " $\frac{1}{4}$ "	7	8	9	10	12	..	..	..	..	$\frac{3}{16}$ " $\frac{1}{4}$ "
$\frac{1}{4}$ " $\frac{5}{16}$ "	6	7	8	9	10	12	14	16	19	$\frac{1}{4}$ " $\frac{5}{16}$ "
$\frac{5}{16}$ " $\frac{3}{8}$ "	5	6	7	8	9	10	12	14	17	$\frac{5}{16}$ " $\frac{3}{8}$ "
$\frac{3}{8}$ " $\frac{7}{16}$ "	4.5	5	6	7	8	9	10	12	15	$\frac{3}{8}$ " $\frac{7}{16}$ "
$\frac{7}{16}$ " $\frac{1}{2}$ "	4	4.5	5	6	7	8	9	10	13	$\frac{7}{16}$ " $\frac{1}{2}$ "
$\frac{1}{2}$ " $\frac{5}{8}$ "	3.5	4	4.5	5	6	7	8	9	11	$\frac{1}{2}$ " $\frac{5}{8}$ "
$\frac{5}{8}$ " $\frac{3}{4}$ "	3	3.5	4	4.5	5	6	7	8	9	$\frac{5}{8}$ " $\frac{3}{4}$ "
$\frac{3}{4}$ " 1 "	2.5	3	3.5	4	4.5	5	6	7	8	$\frac{3}{4}$ " 1 "
1 or over	2.5	2.5	3	3.5	4	4.5	5	6	7	1 or over

of a 2-in. specimen or is outside the middle third of the gage length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

IV. PERMISSIBLE VARIATIONS IN WEIGHT AND THICKNESS.

10. **Permissible Variations.** The cross-section or weight of each piece of steel shall not vary more than 2.5 per cent from that specified; except in the case of sheared plates, which shall be covered by the following permissible variations. One cubic inch of rolled steel is assumed to weigh 0.2833 lb.

(*a*) *When Ordered to Weight per Square Foot*: The weight of each lot [1] in each shipment shall not vary from the weight ordered more than the amount given in Table I.

(*b*) *When Ordered to Thickness*: The thickness of each plate shall not vary more than 0.01 in. under that ordered.

The overweight of each lot [1] in each shipment shall not exceed the amount given in Table II.

V. FINISH.

11. **Finish.** The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI. MARKING.

12. **Marking.** The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice bars and other small sections shall, when loaded for shipment, be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII. INSPECTION AND REJECTION.

13. **Inspection.** The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

14. **Rejection.** (*a*) Unless otherwise specified, any rejection based on tests made in accordance with Section 4 shall be reported within five working days from the receipt of samples.

(*b*) Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

15. **Rehearing.** Samples tested in accordance with Section 4, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

[1] The term "lot" applied to Table I and Table II means all of the plates of each group width and group weight.

STANDARD SPECIFICATIONS FOR STRUCTURAL STEEL FOR BRIDGES

OF THE

AMERICAN SOCIETY FOR TESTING MATERIALS.

ADOPTED 1901; REVISED 1921.

1. **Steel Castings.** The Standard Specifications for Steel Castings adopted by the American Society for Testing Materials, shall govern the purchase of steel castings for bridges. Unless otherwise specified, Class B castings, medium grade, shall be used.

I. MANUFACTURE.

2. **Process.** The steel shall be made by the open-hearth process.

II. CHEMICAL PROPERTIES AND TESTS.

3. **Chemical Composition.** The steel shall conform to the following requirements as to chemical composition:

	Structural Steel.	Rivet Steel.
Phosphorus, Acid	not over 0.06	not over 0.04 per cent
Phosphorus, Basic	" " 0 04	" " 0.04 "
Sulfur	" " 0.05	" " 0.045 "

4. **Ladle Analyses.** An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus and sulfur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Section 3.

5. **Check Analyses.** Analyses may be made by the purchaser from finished material representing each melt. The phosphorus and sulfur content thus determined shall not exceed that specified in Section 3 by more than 25 per cent.

III. PHYSICAL PROPERTIES AND TESTS.

6. **Tension Tests.** (*a*) The material shall conform to the following requirements as to tensile properties:

Properties Considered.	Structural Steel.	Rivet Steel.
Tensile strength, lb. per sq. in.	55,000–65,000[a]	46,000–56,000
Yield point, min., " "	0.5 tens. str.	0.5 tens. str.
Elongation in 8 in., min., per cent	$\frac{1,500,000^{b}}{\text{Tens. str.}}$	$\frac{1,500,000}{\text{Tens. str.}}$
Elongation in 2 in., " "	22	....

[a] See Paragraph (*b*).
[b] See Section 7.

(*b*) In order to meet the required minimum tensile strength of full-size annealed eyebars, the purchaser may determine the tensile strength to be obtained in specimen tests; the range shall not exceed 14,000 lb. per sq. in., and the maximum shall not exceed 74,000 lb. per sq. in. The material shall conform to the requirements as to physical properties other than that of tensile strength, specified in Sections 6, 7 and 8 (*b*).

(*c*) The yield point shall be determined by the drop of the beam of the testing machine.

7. **Modifications in Elongation.** (*a*) For structural steel over ¾ in. in thickness or diameter,

a deduction from the percentage of elongation in 8 in. specified in Section 6 (*a*) of 0.25 per cent shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness or diameter above $\frac{3}{4}$ in., to a minimum of 18 per cent.

(*b*) For structural steel under $\frac{5}{16}$ in. in thickness or diameter, a deduction from the percentage of elongation in 8 in. specified in Section 6 (*a*) of 1.25 per cent shall be made for each decrease of $\frac{1}{32}$ in. in thickness or diameter below $\frac{5}{16}$ in.

8. **Bend Tests.** (*a*) The test specimen for plates, shapes, and bars, except as specified in Paragraphs (*b*), (*c*) and (*d*), shall bend cold through 180 deg. without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or under in thickness, flat on itself; for material over $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material over $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

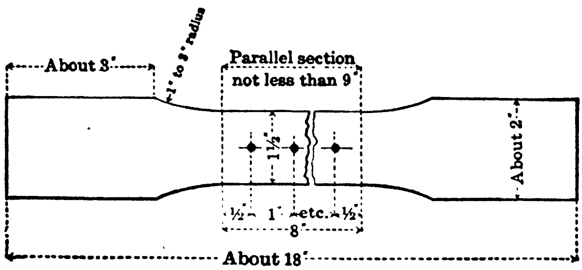
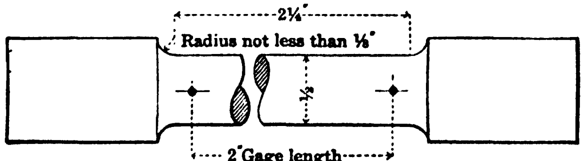


FIG. 1.

(*b*) The test specimen for eyebar flats shall bend cold through 180 deg. without cracking on the outside of the bent portion as follows: For material $\frac{3}{4}$ in. or under in thickness, around a pin the diameter of which is equal to the thickness of the specimen; for material over $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen; and for material over $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to three times the thickness of the specimen.

(*c*) The 1 by $\frac{1}{2}$-in. test specimen for pins, rollers and other bars, when prepared as specified in Section 9, shall bend cold through 180 deg. around a pin 1 in. in diameter without cracking on the outside of the bent portion.

(*d*) The test specimen for rivet steel shall bend cold through 180° flat on itself without cracking on the outside of the bent portion.



NOTE:—The gage length, parallel portions and fillets shall be as shown, but the ends may be of any form which will fit the holders of the testing machine.

FIG. 2.

9. **Test Specimens.** (*a*) Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed; in which case the test specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*b*) Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*d*), (*e*) and (*f*) shall be of the full thickness or diameter of material as rolled.

(*c*) Test specimens for plates, shapes and flats may be machined to the form and dimensions shown in Fig. 1, or with both edges parallel; except that bend test specimens for eyebar flats may have three rolled sides.

(*d*) Tension test specimens for plates and eyebar flats over $1\frac{1}{2}$ in. in thickness, and bend test specimens for plates over $1\frac{1}{2}$ in. in thickness may be machined to a thickness or diameter of at least $\frac{3}{4}$ in. for a length of at least 9 in.

(*e*) Test specimens for bars over $1\frac{1}{2}$ in. in thickness or diameter may be machined to a thickness or diameter of at least $\frac{3}{4}$ in. for a length of at least 9 in.; or tension test specimens may conform to the dimensions shown in Fig. 2, in which case the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend test specimens may be 1 by $\frac{1}{2}$ in. in section.

(*f*) Tension test specimens for pins and rollers shall conform to the dimensions shown in Fig. 2. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend test specimens shall be 1 by $\frac{1}{2}$ in. in section.

(*g*) The tension test specimen shown in Fig. 2 and the 1 by $\frac{1}{2}$-in. bend test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars over $1\frac{1}{2}$ in. in thickness or diameter, midway between the center and surface.

(*h*) The machined sides of rectangular bend test specimens may have the corners rounded to a radius not over $\frac{1}{16}$ in.

(*i*) Test specimens for rivet bars which have been cold drawn shall be normalized before testing.

10. **Number of Tests.** (*a*) One tension and one bend test shall be made from each melt; except that if material from one melt differs $\frac{3}{8}$ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 6 (*a*) and any part of the fracture is more than $\frac{3}{4}$ in. from the center of the gage length of a 2-in. specimen or is outside the middle third of the gage length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

IV. PERMISSIBLE VARIATIONS IN WEIGHT AND THICKNESS.

11. **Permissible Variations.** The cross-section or weight of each piece of steel shall not vary more than 2.5 per cent from that specified; except in the case of sheared plates, which shall be covered by the following permissible variations. One cubic inch of rolled steel is assumed to weigh 0.2833 lb.

(*a*) *When Ordered to Weight per Square Foot*: The weight of each lot[1] in each shipment shall not vary from the weight ordered more than the amount given in Table I, see p. 615.

(*b*) *When Ordered to Thickness*: The thickness of each plate shall not vary more than 0.01 in. under that ordered.

The overweight of each lot[2] in each shipment shall not exceed the amount given in Table II, see p. 615.

V. FINISH.

12. **Finish.** The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI. MARKING.

13. **Marking.** The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice bars and other small sections shall, when loaded for shipment, be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII. INSPECTION AND REJECTION.

14. **Inspection.** The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

15. **Rejection.** (*a*) Unless otherwise specified, any rejection based on tests made in accordance with Section 5 shall be reported within five working days from the receipt of samples.

(*b*) Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

16. **Rehearing.** Samples tested in accordance with Section 5, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

STANDARD SPECIFICATIONS FOR STRUCTURAL NICKEL STEEL

OF THE

AMERICAN SOCIETY FOR TESTING MATERIALS.

ADOPTED AUGUST 25, 1912; REVISED 1921.

I. MANUFACTURE.

1. **Process.** The steel shall be made by the open-hearth process.

2. **Discard.** A sufficient discard shall be made from each ingot intended for eye-bars to secure freedom from injurious piping and undue segregation.

II. CHEMICAL PROPERTIES AND TESTS.

3. **Chemical Composition.** The steel shall conform to the following requirements as to chemical composition:

		Structural Steel.	Rivet Steel.
Carbon		not over 0.45	not over 0.30 per cent
Manganese		" " 0.70	" " 0.60 "
Phosphorus	Acid	" " 0.05	" " 0.04 "
	Basic	" " 0.04	" " 0.03 "
Sulphur		" " 0.04	" " 0.045 "
Nickel		not under 3.25	not under 3.25 "

4. **Ladle Analyses.** An analysis shall be made by the manufacturer from a test ingot taken during the pouring of each melt. A copy of this analysis shall be given to the purchaser or his representative. This analysis shall conform to the requirements specified in Section 3.

5. **Check Analyses.** A check analysis may be made by the purchaser from finished material representing each melt, and this analysis shall conform to the requirements specified in Section 3.

III. PHYSICAL PROPERTIES AND TESTS.

6. **Tension Tests.** (*a*) The steel shall conform to the following requirements as to tensile properties:

TENSILE PPOPERTIES FROM SPECIMEN TESTS.

Properties Considered.	Rivets.	Plates, Shapes and Bars.	Eye-Bars and Rollers,[c] Unannealed.	Eye-Bars[a] and Pins,[c] Annealed.
Tensile strength, lb. per sq. in.	70,000–80,000	85,000–100,000	95,000–110,000	90,000–105,000
Yield point, min., lb. per sq. in.	45,000	50,000	55,000	52,000
Elongation in 8 in., min., per cent.	$\frac{1,500,000}{\text{Tens. Str.}}$	$\frac{1,500,000^{b}}{\text{Tens. Str.}}$	$\frac{1,500,000^{b}}{\text{Tens. Str.}}$	20
Elongation in 2 in., min., per cent.	....	....	16	20
Reduction of area min., per cent.	40	25	25	35

[a] Tests of annealed specimens of eye-bars shall be made for information only.

[b] See Section 7.

[c] Elongation shall be measured in 2 in.

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

7. **Modifications in Elongation.** For plates, shapes, and unannealed bars over 1 in. in thickness, a deduction from the percentage of elongation specified in Section 6 (*a*) of 0.25 per cent shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness above 1 in., to a minimum of 14 per cent.

8. **Character of Fracture.** All broken tension test specimens shall show either a silky or a very fine granular fracture, of uniform color, and free from coarse crystals.

9. **Bend Tests.** (*a*) The test specimen for plates, shapes and bars shall bend cold through 180 deg. without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or

under in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material over $\frac{3}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*) The test specimen for pins and rollers shall bend cold through 180 deg. around a pin 1 in. in diameter without cracking on the outside of the bent portion.

(*c*) The test specimen for rivet steel shall bend cold through 180 deg. flat on itself without cracking on the outside ot the bent portion.

10. **Drifts Tests.** Punched rivet holts pitched two diameters from a planed edge shall stand drifting until the diameter is enlarged 50 per cent, without cracking the metal.

11. **Test Specimens.** (*a*) Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed; in which case the test specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*b*) Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*d*), (*e*) and (*f*) shall be of the full thickness or diameter of material as rolled.

(*c*) Test specimens for plates, shapes and flats may be machined to the form and dimensions shown in Fig. 1, or with both edges parallel; except that bend test specimens for eyebar flats may have three rolled sides. (For Fig. 1, see p. 618.)

(*d*) Tension test specimens for plates and eyebar flats over $1\frac{1}{2}$ in. in thickness, and bend test specimens for plates over $1\frac{1}{2}$ in. in thickness may be machined to a thickness or diameter of at least $\frac{3}{4}$ in. for a length of at least 9 in.

(*e*) Test specimens for bars over $1\frac{1}{2}$ in. in thickness or diameter may be machined to a thickness or diameter of at least $\frac{3}{4}$ in. for a length of at least 9 in.; or tension test specimens may conform to the dimensions shown in Fig. 2, in which case the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend test specimens may be 1 by $\frac{1}{2}$ in. in section. (For Fig. 2, see p. 618.)

(*f*) Tension test specimens for pins and rollers shall conform to the dimensions shown in Fig. 2. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend test specimens shall be 1 by $\frac{1}{2}$ in. in section.

(*g*) The tension test specimen shown in Fig. 2 and the 1 by $\frac{1}{2}$-in. bend test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars over $1\frac{1}{2}$ in. in thickness or diameter, midway between the center and surface.

(*h*) The machined sides of rectangular bend test specimens may have the corners rounded to a radius not over $\frac{1}{16}$ in.

(*i*) Test specimens for rivet bars which have been cold drawn shall be normalized before testing.

12. **Number of Tests.** (*a*) One tension and one bend test shall be made from each melt; except that if material from one melt differs $\frac{3}{8}$ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 6 (*a*) and any part of the fracture is more than $\frac{3}{4}$ in. from the center of the gage length of a 2-in. specimen or is outside the middle third of the gage length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

IV. PERMISSIBLE VARIATIONS IN WEIGHT AND THICKNESS.

13. **Permissible Variations.** The cross-section or weight of each piece of steel shall not vary more than 2.5 per cent from that specified; except in the case of sheared plates, which shall be covered by the following permissible variations. One cubic inch or rolled steel is assumed to weigh 0.2833 lb.

(*a*) *When Ordered to Weight per Square Foot*: The weight of each lot [1] in each shipment shall not vary from the weight ordered more than the amount given in Table I, see p. 615.

(*b*) *When Ordered to Thickness*: The thickness of each plate shall not vary more than 0.01 in. under that ordered.

The overweight of each lot [2] in each shipment shall not exceed the amount given in Table II, see p. 615.

V. FINISH.

14. **Finish.** The finished material shall be free from injurious defects and shall have a workmanlike finish.

[1] The term " lot " applied to Table I means all of the plates of each group width and group weight.

[2] The term " lot " applied to Table II means all of the plates of each group width and group thickness.

VI. MARKING.

15. **Marking.** The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice bars and other small sections shall, when loaded for shipment, be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII. INSPECTION.

16. **Inspection.** The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

17. **Rejection.** (*a*) Unless otherwise specified, any rejection based on tests made in accordance with Section 5 shall be reported within five working days from the receipt of samples.

(*b*) Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected and the manufacturer shall be notified.

18. **Rehearing.** Samples tested in accordance with Section 5, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

VIII. FULL SIZE TESTS.

19. **Tests of Eye-Bars.** (*a*) Full size tests of annealed eye-bars shall conform to the following requirements as to tensile properties:

Tensile strength, lb. per. sq. in.	85,000–100,000
Yield point, min., lb. per sq. in.	48,000
Elongation in 18 ft., min., per cent	10
Reduction of area, min., per cent.	30

(*b*) The yield point shall be determined by the halt of the gage of the testing machine.

STANDARD SPECIFICATIONS FOR BOILER RIVET STEEL

OF THE

AMERICAN SOCIETY FOR TESTING MATERIALS.

ADOPTED AUGUST 25, 1913; REVISED 1921.

A. Requirements for Rolled Bars.

I. MANUFACTURE.

1. **Process.** The steel shall be made by the open-hearth process.

II. CHEMICAL PROPERTIES AND TESTS.

2. **Chemical Composition.** The steel shall conform to the following requirements as to chemical composition:

Manganese	0.30–0.50 per cent
Phosphorus	not over 0.04 "
Sulphur	" " 0.045 "

3. **Ladle Analyses.** An analysis to determine the percentages of carbon, manganese, phosphorus and sulphur shall be made by the manufacturer from a test ingot taken during the pouring of each melt, a copy of which shall be given to the purchaser or his representative. This analysis shall conform to the requirements specified in Section 2.

4. **Check Analyses.** A check analysis may be made by the purchaser from finished material representing each melt, and this analysis shall conform to the requirements specified in Section 2.

III. PHYSICAL PROPERTIES AND TESTS.

5. **Tension Tests.** (*a*) The bars shall conform to the following requirements as to tensile properties:

Tensile strength, lb. per sq. in.	45,000–55,000
Yield point, min., lb. per sq. in.	0.5 tens. str.
Elongation in 8 in., min., per cent	$\frac{1,500,000}{\text{Tens. str.}}$
	(But need not exceed 30 per cent)

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

6. **Bend Tests.** (*a*) *Cold-bend Tests.*—The test specimen shall bend cold through 180 deg. flat on itself without cracking on the outside of the bent portion.

(*b*) *Quench-bend Tests.*—The test specimen, when heated to a light cherry red as seen in the dark (not less than 1200° F.), and quenched at once in water the temperature of which is between 80° and 90° F., shall bend through 180° flat on itself without cracking on the outside of the bent portion.

7. **Test Specimens.** (*a*) Test specimens shall be of the full-size section of material as rolled.

(*b*) Tension and bend test specimens for rivet bars which have been cold drawn shall be normalized before testing.

8. **Number of Tests.** (*a*) Two tension, two cold-bend, and two quench-bend tests shall be made from each melt, each of which shall conform to the requirements specified.

(*b*) If any test specimen develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 5 (*a*) and any part of the fracture is outside the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

IV. PERMISSIBLE VARIATIONS IN GAGE.

9. **Permissible Variations.** The gage of each bar shall not vary more than 0.01 in. from that specified.

V. WORKMANSHIP AND FINISH.

10. **Workmanship.** The finished bars shall be circular within 0.01 in.

11. **Finish.** The finished bars shall be free from injurious defects, and shall have a workmanlike finish.

VI. MARKING.

12. **Marking.** Rivet bars shall, when loaded for shipment, be properly separated and marked with the name or brand of the manufacturer and the melt number for identification. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII. INSPECTION AND REJECTION.

13. **Inspection.** The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the bars ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the bars are being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

14. **Rejection.** (*a*) Unless otherwise specified, any rejection based on tests made in accordance with Section 4 shall be reported within five working days from the receipt of samples.

(*b*) Bars which show injurious defects subsequent to their acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

15. **Rehearing.** Samples tested in accordance with Section 4, which represent rejected bars, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

B. Requirements for Rivets.

I. PHYSICAL PROPERTIES AND TESTS.

16. **Tension Tests.** The rivets, when tested, shall conform to the requirements as to tensile properties specified in Section 5, except that the elongation shall be measured on a gage length not less than four times the diameter of the rivet.

17. **Bend Tests.** The rivet shank shall bend cold through 180 degrees flat on itself without cracking on the outside of the bent portion.

18. **Flattening Tests.** The rivet heads shall flatten, while hot, to a diameter 2½ times the diameter of the shank without cracking at the edges.

19. (*a*) When specified, one tension test shall be made from each size in each lot of rivets offered for inspection.

(b) Three bend and three flattening tests shall be made from each size in each lot of rivets offered for inspection, each of which shall conform to the requirements specified.

II. WORKMANSHIP AND FINISH.

20. **Workmanship.** Rivets shall be true to form, concentric, and shall be made in a workmanlike manner.

21. **Finish.** The finished rivets shall be free from injurious defects.

III. INSPECTION AND REJECTION.

22. **Inspection.** The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the rivets ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the rivets are being furnished in accordance with these specifications. All tests and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

23. **Rejection.** Rivets which show injurious defects subsequent to their acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

STANDARD SPECIFICATIONS FOR BILLET-STEEL REINFORCEMENT BARS OF THE AMERICAN SOCIETY FOR TESTING MATERIALS.

ADOPTED AUGUST 25, 1913; REVISED 1914.

1. **Classes.** (a) These specifications cover three classes of billet-steel concrete reinforcement bars, namely: plain, deformed, and cold-twisted.

(b) Plain and deformed bars are of three grades, namely: structural-steel, intermediate and hard.

2. **Basis of Purchase.** (a) The structural-steel grade shall be used unless otherwise specified.

(b) If desired, cold-twisted bars may be purchased on the basis of tests of the hot-rolled bars before twisting, in which case such tests shall govern and shall conform to the requirements specified for plain bars of structural steel grade.

I. MANUFACTURE.

3. **Process.** (a) The steel may be made by the Bessemer or the open hearth process.

(b) The bars shall be rolled from new billets. No rerolled material will be accepted.

4. **Cold-twisted Bars.** Cold-twisted bars shall be twisted cold with one complete twist in a length not over 12 times the thickness of the bar.

II. CHEMICAL PROPERTIES AND TESTS.

5. **Chemical Composition.** The steel shall conform to the following requirements as to chemical composition:

Phosphorus	Bessemer.........	not over 0.10 per cent
	Open-hearth.......	" " 0.05 "

6. **Ladle Analyses.** An analysis to determine the percentage of carbon, manganese, phosphorus and sulphur, shall be made by the manufacturer from a test ingot taken during the pouring of each melt, a copy of which shall be given to the purchaser or his representative. This analysis shall conform to the requirements specified in Section 5.

7. **Check Analyses.** Analyses may be made by the purchaser from finished bars representing each melt of open-hearth steel, and each melt, or lot of ten tons, of Bessemer steel, in which case an excess of 25 per cent above the requirements specified in Section 5 shall be allowed.

III. PHYSICAL PROPERTIES AND TESTS.

8. **Tension Tests.** (a) The bars shall conform to the following requirements as to tensile properties:

TENSILE PROPERTIES.

Properties Considered.	Plain Bars.			Deformed Bars.			Cold-twisted Bars.
	Structural-Steel Grade.	Intermediate Grade.	Hard Grade.	Structural-Steel, Grade.	Intermediate Grade.	Hard Grade.	
Tensile strength, lb. per sq. in....	55,000 to 70,000	70,000 to 85,000	80,000 min.	55,000 to 70,000	70,000 to 85,000	80,000 min.	Recorded only
Yield point, min., lb. per sq. in.....	33,000	40,000	50,000	33,000	40,000	50,000	55,000
Elongation in 8 in., min., per cent....	$\frac{1,400,000^a}{\text{Tens. str.}}$	$\frac{1,300,000^a}{\text{Tens. str.}}$	$\frac{1,200,000^a}{\text{Tens. str.}}$	$\frac{1,250,000^a}{\text{Tens. str.}}$	$\frac{1,125,000^a}{\text{Tens. str.}}$	$\frac{1,000,000^a}{\text{Tens. str.}}$	5

[a] See Section 9.

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

9. **Modifications in Elongation.** (*a*) For plain and deformed bars over $\frac{3}{4}$ in. in thickness or diameter, a deduction from the percentages of elongation specified in Section 8 (*a*) of 0.25 per cent shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness or diameter above $\frac{3}{4}$ in.

(*b*) For plain and deformed bars under $\frac{7}{16}$ in. in thickness or diameter, a deduction from the percentages of elongation specified in Section 8 (*a*) of 0.5 per cent shall be made for each decrease of $\frac{1}{32}$ in. of the specified thickness or diameter below $\frac{7}{16}$ in.

10. **Bend Tests.** The test specimen shall bend cold around a pin without cracking on the outside of the bent portion, as follows:

BEND-TEST REQUIREMENTS.

Thickness or Diameter of Bar.	Plain Bars.			Deformed Bars.			Cold-twisted Bars.
	Structural-Steel Grade.	Intermediate Grade.	Hard Grade.	Structural-Steel Grade.	Intermediate Grade.	Hard Grade.	
Under $\frac{3}{4}$ in......	180 deg. d = t	180 deg. d = 2t	180 deg. d = 3t	180 deg. d = t	180 deg. d = 3t	180 deg. d = 4t	180 deg d = 2t
$\frac{3}{4}$ in. or over.....	180 deg. d = t	90 deg. d = 2t	90 deg. d = 3t	180 deg. d = 2t	90 deg. d = 3t	90 deg. d = 4t	180 deg d = 3t

Explanatory Note: d = the diameter of pin about which the specimen is bent;
t = the thickness or diameter of the specimen.

11. **Test Specimens.** (*a*) Tension and bend test specimens for plain and deformed bars shall be taken from the finished bars, and shall be of the full thickness or diameter of material as rolled; except that the specimens for deformed bars may be machined for a length of at least 9 in., if deemed necessary by the manufacturer to obtain uniform cross-section.

(*b*) Tension and bend test specimens for cold-twisted bars shall be taken from the finished bars, without further treatment; except as specified in Section 2 (*b*).

12. **Number of Tests.** (*a*) One tension and one bend test shall be made from each melt of open-hearth steel, and from each melt, or lot of ten tons, of Bessemer steel; except that if material from one melt differs $\frac{3}{8}$ in. or more in thickness or diameter, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*) If any test specimen shows defective machining or develops flaws, or if a tension test specimen breaks outside the middle third of the gage length, it may be discarded and another specimen substituted.

IV. PERMISSIBLE VARIATIONS IN WEIGHT.

13. **Permissible Variations.** The weight of any lot of bars shall not vary more than 5 per cent from the theoretical weight of that lot.

V. FINISH.

14. **Finish.** The finished bars shall be free from injurious defects and shall have a workmanlike finish.

VI. INSPECTION AND REJECTION.

15. **Inspection.** The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the bars ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the bars are being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

16. **Rejection.** (*a*) Unless otherwise specified, any rejection based on tests made in accordance with Section 7 shall be reported within five working days from the receipt of samples.

(*b*) Bars which show injurious defects subsequent to their acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

17. **Rehearing.** Samples tested in accordance with Section 7, which represent rejected bars, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

STANDARD SPECIFICATIONS FOR RAIL-STEEL REINFORCEMENT BARS

OF THE

AMERICAN SOCIETY FOR TESTING MATERIALS.

ADOPTED AUGUST 25, 1913; REVISED 1914.

1. **Classes.** These specifications cover three classes of rail-steel concrete reinforcement bars, namely: plain, deformed, and hot-twisted.

I. MANUFACTURE.

2. **Process.** The bars shall be rolled from standard section Tee rails.

3. **Hot-twisted Bars.** Hot-twisted bars shall have one complete twist in a length not over 12 times the thickness of the bar.

II. PHYSICAL PROPERTIES AND TESTS.

4. **Tension Tests.** (*a*) The bars shall conform to the following minimum requirements as to tensile properties:

Properties Considered.	Plain Bars.	Deformed and Hot-twisted Bars.
Tensile strength, lb. per sq. in.	80,000	80,000
Yield point, lb. per sq. in.	50,000	50,000
Elongation in 8 in., per cent[1]	$\frac{1,200,000}{\text{Tens. str.}}$	$\frac{1,000,000}{\text{Tens. str.}}$

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

5. **Modifications in Elongation.** (*a*) For bars over $\frac{3}{4}$ in. in thickness or diameter, a deduction of 1 from the percentages of elongation specified in Section 4 (*a*) shall be made for each increase of $\frac{1}{8}$ in. in thickness or diameter above $\frac{3}{4}$ in.

(*b*) For bars under $\frac{7}{16}$ in. in thickness or diameter, a deduction of 1 from the percentages of elongation specified in Section 4 (*a*) shall be made for each decrease of $\frac{1}{16}$ in. in thickness or diameter below $\frac{7}{16}$ in.

6. **Bend Tests.** The test specimen shall bend cold around a pin without cracking on the outside of the bent portion, as follows:

[1] See Section 5.

Thickness or Diameter of Bar.	Plain Bars.	Deformed and Hot-twisted Bars.
Under ¾ in.	180 deg. d = 3 t	180 deg. d = 4 t
¾ in. or over	90 deg. d = 3 t	90 deg. d = 4 t

EXPLANATORY NOTE: d = the diameter of pin about which the specimen is bent;
t = the thickness or diameter of the specimen.

7. **Test Specimens.** (*a*) Tension and bend test specimens for plain and deformed bars shall be taken from the finished bars, and shall be of the full thickness or diameter of bars as rolled; except that the specimens for deformed bars may be machined for a length of at least 9 in., if deemed necessary by the manufacturer to obtain uniform cross-section.

(*b*) Tension and bend test specimens for hot-twisted bars shall be taken from the finished bars, without further treatment.

8. **Number of Tests.** (*a*) One tension and one bend test shall be made from each lot of ten tons or less of each size of bar rolled from rails varying not more than 10 lb. per yd. in nominal weight.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 4 (*a*) and any part of the fracture is outside the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

III. PERMISSIBLE VARIATIONS IN WEIGHT.

9. **Permissible Variations.** The weight of any lot of bars shall not vary more than 5 per cent from the theoretical weight of that lot.

IV. FINISH.

10. **Finish.** The finished bars shall be free from injurious defects and shall have a workmanlike finish.

V. INSPECTION AND REJECTION.

11. **Inspection.** The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the bars ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the bars are being furnished in accordance with these specifications. All tests and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

12. **Rejection.** Bars which show injurious defects subsequent to their acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

STANDARD SPECIFICATIONS FOR STEEL CASTINGS

OF THE

AMERICAN SOCIETY FOR TESTING MATERIALS

ADOPTED AUGUST 25, 1913.

1. **Classes.** These specifications cover two classes of castings, namely:

Class A, ordinary castings for which no physical requirements are specified;

Class B, castings for which physical requirements are specified. These are of three grades: hard, medium, and soft.

2. **Patterns.** (*a*) Patterns shall be made so that sufficient finish is allowed to provide for all variations in shrinkage.

(*b*) Patterns shall be painted three colors to represent metal, cores, and finished surfaces. It is recommended that core prints shall be painted black and finished surfaces red.

3. **Basis of Purchase.** The purchaser shall indicate his intention to substitute the test to destruction specified in Section 11 for the tension and bend tests, and shall designate the patterns from which castings for this test shall be made.

I. MANUFACTURE.

4. **Process.** The steel shall be made by one or more of the following processes: open-hearth, electric furnace, side blow converter or crucible.

5. **Heat Treatment.** (*a*) Class A castings need not be annealed unless so specified.

(*b*) Class B castings shall be properly annealed, the treatment depending upon the design and chemical composition of the castings.

II. CHEMICAL PROPERTIES AND TESTS.

6. **Chemical Composition.** The castings shall conform to the following requirements as to chemical composition:

	CLASS A.	CLASS B.
Carbon	not over 0.30 per cent	....
Phosphorus	" " 0.06 "	not over 0.05 per cent
Sulphur	....	" " 0.05 "

7. **Ladle Analyses.** An analysis to determine the percentages of carbon, manganese, phosphorus and sulphur shall be made by the manufacturer from a test ingot taken during the pouring of each melt, a copy of which shall be given to the purchaser or his representative. This analysis shall conform to the requirements specified in Section 6. Drillings for analysis shall be taken not less than ¼ in. beneath the surface of the test ingot.

8. **Check Analyses.** (*a*) Analyses of Class A castings may be made by the purchaser, in which case an excess of 20 per cent above the requirement as to phosphorus specified in Section 6 shall be allowed. Drillings for analysis shall be taken not less than ¼ in. beneath the surface.

(*b*) Analyses of Class B castings may be made by the purchaser from a broken tension or bend test specimen, in which case an excess of 20 per cent above the requirements as to phosphorus and sulphur specified in Section 6 shall be allowed. Drillings for analysis shall be taken not less than ¼ in. beneath the surface.

III. PHYSICAL PROPERTIES AND TESTS.

(FOR CLASS B CASTINGS ONLY.)

9. **Tension Tests.** (*a*) The castings shall conform to the following minimum requirements as to tensile properties:

	HARD.	MEDIUM.	SOFT.
Tensile strength, lb. per sq. in.	80 000	70 000	60 000
Yield point, lb. per sq. in.	0.45 tens.str.	0.45 tens.str.	0.45 tens.str.
Elongation in 2 in., per cent	15	18	22
Reduction of area, "	20	25	30

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

10. **Bend Tests.** (*a*) The test specimen for soft castings shall bend cold through 120 deg., and for medium castings through 90 deg., around a 1-in. pin, without cracking on the outside of the bent portion.

(*b*) Hard castings shall not be subject to bend test requirements.

11. **Alternative Tests to Destruction.** In the case of small or unimportant castings, a test to destruction on three castings from a lot may be substituted for the tension and bend tests. This

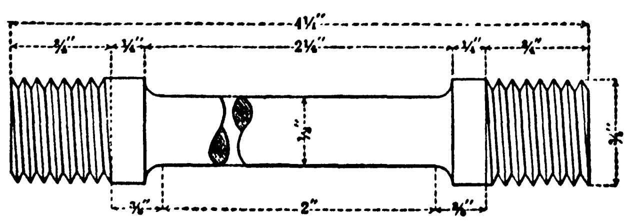


FIG. 1.

test shall show the material to be ductile, free from injurious defects, and suitable for the purpose intended. A lot shall consist of all castings from one melt, in the same annealing charge.

12. **Test Specimens.** (*a*) Sufficient test bars, from which the test specimens required in Section 13 (*a*) may be selected, shall be attached to castings weighing 500 lb. or over, when the

design of the castings will permit. If the castings weigh less than 500 lb., or are of such a design that test bars cannot be attached, two test bars shall be cast to represent each melt; or the quality of the castings shall be determined by tests to destruction as specified in Section 11. All test bars shall be annealed with the castings they represent.

(*b*) The manufacturer and purchaser shall agree whether test bars can be attached to castings, on the location of the bars on the castings, on the castings to which bars are to be attached, and on the method of casting unattached bars.

(*c*) Tension test specimens shall be of the form and dimensions shown in Fig. 1. Bend test specimens shall be machined to 1 by $\frac{1}{2}$ in. in section with corners rounded to a radius not over $\frac{1}{16}$ in.

13. **Number of Tests.** (*a*) One tension and one bend test shall be made from each annealing charge. If more than one melt is represented in an annealing charge, one tension and one bend test shall be made from each melt.

(*b*) If any test specimen shows defective machining or develops flaws, or if a tension test specimen breaks outside the gage length, it may be discarded; in which case the manufacturer and the purchaser or his representative shall agree upon the selection of another specimen in its stead.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 9 (*a*) and any part of the fracture is more than $\frac{3}{4}$ in. from the center of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

13a. **Retests.** If the results of the physical tests of any test lot do not conform to the requirements specified, the manufacturer may re-anneal such lot not more than twice and retests shall be as specified in Sections 9 and 10.

IV. WORKMANSHIP AND FINISH.

14. **Workmanship.** The castings shall substantially conform to the sizes and shapes of the patterns, and shall be made in a workmanlike manner.

15. **Finish.** (*a*) The castings shall be free from injurious defects.

(*b*) Minor defects which do not impair the strength of the castings may, with the approval of the purchaser or his representative, be welded by an approved process. The defects shall first be cleaned out to solid metal; and after welding, the castings shall be annealed, if specified by the purchaser or his representative.

(*c*) The castings offered for inspection shall not be painted or covered with any substance that will hide defects, nor rusted to such an extent as to hide defects.

V. INSPECTION AND REJECTION.

16. **Inspection.** The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the castings ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the castings are being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

17. **Rejection.** (*a*) Unless otherwise specified, any rejection based on tests made in accordance with Section 8 shall be reported within five working days from the receipt of samples.

(*b*) Castings which show injurious defects subsequent to their acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

18. **Rehearing.** Samples tested in accordance with Section 8, which represent rejected castings, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

VI. SPECIAL REQUIREMENTS FOR CASTINGS FOR SHIPS.

19. **Castings for Ships.** In addition to the preceding requirements, castings for ships, when so specified, shall conform to the following requirements:

20. **Heat Treatment.** All castings shall be annealed.

21. **Number of Tests.** (*a*) One tension and one bend test shall be made from each of the following castings: stern frames, stern posts, twin screw spectacle frames, propellor shaft brackets, rudders, steering quadrants, tillers, stems, anchors, and other castings when specified.

(*b*) When a casting is made from more than one melt, four tension and four bend tests shall be made from each casting.

22. **Percussion Tests.** (*a*) A percussion test shall be made on each of the following castings: stern frames, stern posts, twin screw spectacle frames, propellor shaft brackets, rudders, steering quadrants, tillers, stems, anchors, and other castings when specified.

(*b*) For this test, the casting shall be suspended by chains and hammered all over with a hammer of a weight approved by the purchaser or his representative. If cracks, flaws, defects, or weakness appear after such treatment, the casting will be rejected.

CORROSION OF IRON AND STEEL.—If iron or steel is left exposed to the atmosphere it unites with oxygen and water to form rust. Where the metal is further exposed to the action of corrosive gases the rate of rusting is accelerated but the action is similar to that of ordinary rusting. Neither dry air nor water free from oxygen has any corrosive effect. While not essential to corrosion acids greatly hasten its action. It seems evident that some weak electrolysis is essential for corrosive action. Where iron or steel are in contact with water electrolytic action will always take place, although the amount is very small under ordinary conditions. Where a considerable electrolytic force exists the corrosion is greatly hastened. The increase in the use of electricity has doubtless had a tendency to increase the corrosion of iron and steel and to make the problem of the preservation of iron and steel from corrosion of great importance.

In an article on "The Corrosion of Iron" in Proceedings of American Society for Testing Materials, vol. VII, 1907, pages 211 to 228, Mr. Allerson S. Cushman shows that the two factors without which the corrosion of iron is impossible are electrolysis and the presence of hydrogen in the electrolyzed or "ionic" condition. The electrolytic action can only take place in the presence of oxygen or some other oxidizing agent. Rust is a hydroxide of iron—ferric hydroxide, FeO_3H_3. The corrosion of iron or steel may be prevented or retarded by covering it with a coating that will protect it from the water or the air.

It is commonly believed, with good reason, that cast iron corrodes less rapidly than either wrought iron or steel. The graphite in the cast iron and the silicious coating that the cast iron receives in molding doubtless assist in protecting the cast iron from corrosion.

It is also commonly believed that steel corrodes more rapidly than wrought iron. The tests that have been made to determine the relative corrosion of wrought iron and steel are very conflicting, but it appears certain that the difference in the corrosion of well made steel and well made wrought iron is very slight. The acid test as a measure of natural corrosion has been used, especially by firms manufacturing and selling "ingot iron" (very low carbon Bessemer or open-hearth steel). Committee A-5 on the Corrosion of Iron and Steel of the American Society for Testing Materials in the Proceedings of the Society, vol. XI, 1911, page 100, states *that it considers the acid test as unreliable as a measure of natural corrosion and does not recommend its use.*

In the paper on "The Corrosion of Iron" above referred to, Mr. Cushman states:—"A very widespread impression prevails that charcoal iron or a puddled wrought iron are more resistant to corrosion than steel manufactured by the Bessemer and open-hearth processes. It is by no means certain that this is the case, but it would follow from the electrolytic theory that in order to have the highest resistance to corrosion a metal should either be as free as possible from certain impurities, such as manganese, or should be so homogeneous as not to retain localized positive and negative nodes for a long time without change. Under the first condition iron would appear to have the advantage, but under the second much would depend upon the care exercised in manufacture, whatever process was used."

From the preceding discussion it would appear that neither "ingot iron" nor wrought iron has any advantage in resisting corrosion over a well made structural steel.

PAINT.*—The paints in use for protecting structural steel may be divided into oil paints, tar paints, asphalt paints, varnishes, lacquers, and enamel paints. The last two mentioned are too expensive for use on a large scale and will not be considered.

OIL PAINTS.—An oil paint consists of a drying oil or varnish and a pigment, thoroughly mixed together to form a workable mixture. "A good paint is one that is readily applied, has good covering powers, adheres well to the metal, and is durable." The pigment should be inert to the metal to which it is applied and also to the oil with which it is mixed. Linseed oil is commonly used as the varnish or vehicle in oil paints, and is unsurpassed in durability by any other drying oil. Pure linseed oil will, when applied to a metal surface, form a transparent coating that offers considerable protection for a time, but is soon destroyed by abrasion and the action of the elements. To make the coating thicker, harder and more dense, a pigment is added to the oil. An oil paint is analogous to concrete, the linseed oil and pigment in the paint corresponding to the

* This discussion on paints is taken from the author's "The Design of Steel Mill Buildings."

cement and the aggregate in the concrete. The pigments used in making oil paints for protecting metal may be divided into four groups as follows: (1) lead; (2) zinc; (3) iron; (4) carbon.

Linseed Oil.—Linseed oil is made by crushing and pressing flaxseed. The oil contains some vegetable impurities when made, and should be allowed to stand for two or three months to purify and settle before being used. In this form the oil is known as raw linseed oil, and is ready for use. Raw linseed oil dries (oxidizes) very slowly and for that reason is not often used in a pure state for structural iron paint. The rate of drying of raw linseed oil increases with age; an old oil being very much better for paint than that which has been but recently extracted. Raw linseed oil can be made to dry more rapidly by the addition of a drier or by boiling. Linseed oil dries by oxidation and not by evaporation, and therefore any material that will make it take up oxygen more rapidly is a drier. A common method of making a drier for linseed oil is to put the linseed oil in a kettle, heat it to a temperature of 400 to 500 degrees F., and stir in about four pounds of red lead or litharge, or a mixture of the two, to each gallon of oil. This mixture is then thinned down by adding enough linseed oil to make four gallons for each gallon of raw oil first put in the kettle. The addition of four gallons of this drier to forty gallons of raw oil will reduce the time of drying from about five days to twenty-four hours. A drier made in this way costs more than the pure linseed oil, so that driers are very often made by mixing lead or manganese oxide with rosin and turpentine, benzine, or rosin oil. These driers can be made for very much less than the price of good linseed oil, and are used as adulterants; *the more of the drier that is put into the paint, the quicker it will dry and the poorer it becomes.* Japan drier is often used with raw oil, and when this or any other drier is added to raw oil in barrels, the oil is said to be " boiled through the bung hole."

Boiled linseed oil is made by heating raw oil, to which a quantity of red lead, litharge, sugar of lead, etc., has been added, to a temperature of 400 to 500 degrees F., or by passing a current of heated air through the oil. Heating linseed oil to a temperature at which merely a few bubbles rise to the surface makes it dry more rapidly than the unheated oil; however, if the boiling is continued for more than a few hours the rate of drying is decreased by the boiling. Boiled linseed oil is darker in color than raw oil, and is much used for outside paints. It should dry in from 12 to 24 hours when spread out in a thin film on glass. Raw oil makes a stronger and better film than boiled oil, but it dries so slowly that it is seldom used for outside work without the addition of a drier.

Lead.—*White Lead* (hydrated carbonate of lead—specific gravity 6.4) is used for interior and exterior wood work. White lead forms an excellent pigment on account of its high adhesion and covering power, but it is easily darkened by exposure to corrosive gases and rapidly disintegrates under these conditions, requiring frequent renewal. It does not make a good bottom coat for other paints, and if it is to be used at all for metal work it should be used over another paint.

Red Lead (minium; lead tetroxide—specific gravity 8.3) is a heavy, red powder approximating in shade to orange; is affected by acids, but when used as a paint is very stable in light and under exposure to the weather. Red lead is seldom adulterated, about the only substance used for the purpose being red oxide. Red lead is prepared by changing metallic lead into monoxide litharge, and converting this product into minium in calcining ovens. Red lead intended for paints must be free from metallic lead. One ounce of lampblack added to one pound of red lead changes the color to a deep chocolate and increases the time of drying. This compound when mixed in a thick paste will keep 30 days without hardening.

Zinc.—Zinc white (zinc oxide—specific gravity 5.3) is a white loose powder, devoid of smell or taste and has a good covering power. Zinc paint has a tendency to peel, and when exposed there is a tendency to form a zinc soap with the oil which is easily washed off, and it therefore does not make a good paint. However, when mixed with red oxide of lead in the proportions of 1 lead to 3 zinc, or 2 lead to 1 zinc, and ground with linseed oil, it makes a very durable paint for metal surfaces. This paint dries very slowly, the zinc acting to delay hardening about the same as lampblack.

Iron Oxide.—Iron oxide (specific gravity 5) is composed of anhydrous sesquioxide (hematite) and hydrated sesquioxide of iron (iron rust). The anhydrous oxide is the characteristic ingredient of this pigment and very little of the hydrated oxide should be present. Hydrated sesquioxide of iron is simply iron rust, and it probably acts as a carrier of oxygen and accelerates corrosion when it is present in considerable quantities. Mixed with the iron ore are various other ingredients, such as clay, ocher and earthy materials, which often form 50 to 75 per cent of the mass. Brown and dark red colors indicate the anhydrous oxide and are considered the best. Bright red, bright purple and maroon tints are characteristic of hydrated oxide and make less durable paints than the darker tints. Care should be used in buying iron oxide to see that it is finely ground and is free from clay and ocher.

Carbon.—The most common forms of carbon in use for paints are lampblack and graphite. Lampblack (specific gravity 2.6) is a great absorbent of linseed oil and makes an excellent pigment Graphite (black lead or plumbago—specific gravity 2.4) is a more or less impure form of carbon. and when pure is not affected by acids. Graphite does not absorb nor act chemically on linseed

oil, so that the varnish simply holds the particles of pigment together in the same manner as the cement in a concrete. There are two kinds of graphite in common use for paints—the granular and the flake graphite. The Dixon Graphite Co., of Jersey City, uses a flake graphite combined with silica, while the Detroit Graphite Manufacturing Co. uses a mineral ore with a large percentage of graphitic carbon in granulated form. On account of the small specific gravity of the pigment, carbon and graphite paints have a very large covering capacity. The thickness of the coat is, however, correspondingly reduced. Boiled linseed oil should always be used with carbon pigments.

Mixing the Paint.—The pigment should be finely ground and should preferably be ground with the oil. The materials should be bought from reliable dealers, and should be mixed as wanted. If it is not possible to grind the paint, better results will usually be obtained from hand mixed paints made of first class materials than from the ordinary run of prepared paints that are supposed to have been ground. Many ready mixed paints are sold for less than the price of linseed oil, which makes it evident that little if any oil has been used in the paint. The paint should be thinned with oil, or if necessary a small amount of turpentine may be added; *however turpentine is an adulterant and should be used sparingly. Benzine, gasoline, etc., should never be used in paints,* as the paint dries without oxidizing and then rubs off like chalk.

Proportions.—The proper proportions of pigment and oil required to make a good paint vary with the different pigments, and the methods of preparing the paint; the heavier and the more finely ground pigments require less oil than the lighter or coarsely ground while ground paints require less oil than ordinary mixed paints. A common rule for mixing paints ground in oil is to mix with each gallon of linseed oil, dry pigment equal to three to four times the specific gravity of the pigment, the weight of the pigment being given in pounds. This rule gives the following weights of pigment per gallon of linseed oil: white lead, 19 to 26 lb.; red lead, 25 to 33 lb.; zinc, 15 to 21 lb.; iron oxide, 15 to 20 lb.; lampblack, 8 to 10 lb.; graphite, 8 to 10 lb. The weights of pigment used per gallon of oil varies about as follows: red lead, 20 to 33 lb.; iron oxide, 8 to 25 lb.; graphite, 3 to 12 lb.

Covering Capacity.—The covering capacity of a paint depends upon the uniformity and thickness of the coating; the thinner the coating the larger the surface covered per unit of paint. To obtain any given thickness of paint therefore requires practically the same amount of paint whatever its pigment may be. The claims often urged in favor of a particular paint that it has a large covering capacity may mean nothing but that an excess of oil has been used in its fabrication. An idea of the relative amounts of oil and pigment required, and the covering capacity of different paints may be obtained from Table VIII, Chapter XIII.

Light structural work will average about 250 square feet, and heavy structural work about 150 square feet of surface per net ton of metal.

It is the common practice to estimate ½ gallon of paint for the first coat and ⅜ gallon for the second coat per ton of structural steel, for average conditions.

Applying the Paint.—The paint should be thoroughly brushed out with a round brush to remove all the air. The paint should be mixed only as wanted, and should be kept well stirred. When it is necessary to apply paint in cold weather, it should be heated to a temperature of 130 to 150 degrees F.; paint should not be put on in freezing weather. Paint should not be applied when the surface is damp, or during foggy weather. The first coat should be allowed to stand for three or four days, or until thoroughly dry, before applying the second coat. If the second coat is applied before the first coat has dried, the drying of the first coat will be very much retarded.

Cleaning the Surface.—Before applying the paint all scale, rust, dirt, grease and dead paint should be removed. The metal may be cleaned by pickling in an acid bath, by scraping and brushing with wire brushes, or by means of the sand blast. In the process of pickling the metal is dipped in an acid bath, which is followed by a bath of milk of lime, and afterwards the metal is washed clean in hot water. The method is expensive and not satisfactory unless extreme care is used in removing all traces of the acid. Another objection to the process is that it leaves the metal wet and allows rusting to begin before the paint can be applied. The most common method of cleaning is by scraping with wire brushes and chisels. This method is slow and laborious. The method of cleaning by means of a sand blast has been used to a limited extent and promises much for the future. The average cost of cleaning five bridges in Columbus, Ohio, in 1902, was 3 cts. per sq. ft. of surface cleaned.* The bridges were old and some were badly rusted. The painters followed the sand blast and covered the newly cleaned surface with paint before the rust had time to form.

Mr. Lilly estimates the cost of cleaning light bridge work at the shop with the sand blast at $1.75 per ton, and the cost of heavy bridge work at $1.00 per ton. In order to remove the mill scale it has been recommended that rusting be allowed to start before the sand blast is used. One of the advantages of the sand blast is that it leaves the surface perfectly dry, so that the paint can be applied before any rust has formed.

* Sand Blast Cleaning of Structural Steel, by G. W. Lilly, Trans. Am. Soc. C. E., Feb., 1903.

Priming or Shop Coat.—Engineers are very much divided as to what makes the best priming coat; some specify a first coat of pure linseed oil and others a priming coat of paint. Linseed oil makes a transparent coating that allows imperfections in the workmanship and rusted spots to be easily seen; it is not permanent however, and if the metal is exposed for a long time the oil will often be entirely removed before the second coat is applied. It is also claimed that the paint will not adhere as well to linseed oil that has weathered as to a good paint. Linseed oil gives better results if applied hot to the metal. Another advantage of using oil as a priming coat is that the erection marks can be painted over with the oil without fear of covering them up. Red lead paint toned down with lampblack is probably used more for a priming coat than any other paint; the B. & O. R. R. uses 10 oz. of lampblack to every 12 lb. of red lead. Linseed oil mixed with a small amount of lampblack makes a very satisfactory priming or shop coat.

Without going further into the controversy it would seem that there is very little choice between linseed oil and a good red lead paint for a priming coat. For data on the standard shop paints specified by different railroads, see digest of specifications in Chapter IV.

Finishing Coat.—From a careful study of the question of paints, it would seem that for ordinary conditions, the quality of the materials and workmanship is of more importance in painting metal structures than the particular pigment used. If the priming coat has been properly applied there is no reason why any good grade of paint composed of pure linseed oil and a very finely ground, stable and chemically non-injurious pigment will not make a very satisfactory finishing coat. Where the paint is to be subjected to the action of corrosive gases or blasts, however, there is certainly quite a difference in the results obtained with the different pigments. The graphite and asphalt paints appear to withstand the corroding action of smelter and engine gases better than red lead or iron oxide paints; while red lead is probably better under these conditions than iron oxide. Portland cement paint or coal tar paint are the only paints that will withstand the action of engine blasts.

To obtain the best results in painting metal structures therefore, proceed as follows: (1) prepare the surface of the metal by carefully removing all dirt, grease, mill scale, rust, etc., and give it a priming coat of pure linseed oil or a good paint—red lead seems to be the most used for this purpose; (2) after the metal is in place carefully remove all dirt, grease, etc., and apply the finishing coats—preferably not less than two coats—giving ample time for each coat to dry before applying the next. The separate coats of paint should be of different colors. Painting should not be done in rainy weather, or when the metal is damp, nor in cold weather unless special precautions are taken to warm the paint. The best results will usually be obtained if the materials are purchased in bulk from a responsible dealer and the paint ground as wanted. Good results are obtained with many of the patent or ready mixed paints, but it is not possible in this place to go into a discussion of their respective merits.

ASPHALT PAINT.—Many prepared paints are sold under the name of asphalt that are mixtures of coal tar, or mineral asphalt alone, or combined with a metallic base, or oils. The exact compositions of the patent asphalt paints are hard to determine. Black bridge paint made by Edward Smith & Co., New York City, contains asphaltum, linseed oil, turpentine and Kauri gum. The paint has a varnish-like finish and makes a very satisfactory paint. The black shades of asphalt paint are the only ones that should be used.

COAL TAR PAINT.—Coal tar paint is occasionally used for painting gas tanks, smelters, and similar structures that receive rough usage. Coal tar paint mixed as described below has been used by the U. S. Navy Department for painting the hulls of ships. It should give satisfactory service where the metal is subject to corrosion. The coal tar paint is mixed as follows: The proportions of the mixture are slightly variable according to the original consistency of the tar, the use for which it is intended and the climate in which it is used. The proportions will vary between the following proportions in volume.

	Coal Tar.	Portland Cement.	Kerosene Oil.
New Orleans Mixture	8	1	1
Annapolis Mixture	16	4	3

The Portland cement should first be stirred into the kerosene, forming a creamy mixture, the mixture is then stirred into the coal tar. The paint should be freshly mixed and kept well stirred. This paint sticks well, does not run when exposed to the sun's rays and is a very satisfactory paint for rough work. The cost of the paint will vary from 10 to 20 cts. per gallon. The kerosene oil acts as a drier, while the Portland cement neutralizes the coal tar.

If it is desired to paint with oil paint a structure which has been painted with coal tar paint, the surface must be scraped and all the coal tar removed.

CEMENT AND CEMENT PAINT.—Experiments have shown that a thin coating of Portland cement is effective in preventing rust; that a concrete to be effective in preventing rust must be dense and made very wet. The steel must be clean when imbedded in the concrete. There is quite a difference of opinion as to whether the metal should be painted before being imbedded or

not. It is probably best to paint the metal if it is not to be imbedded at once, or is not to be used in concrete-steel construction where the adhesion of the cement to the metal is an essential element. When the metal is to be imbedded immediately it is better not to paint it.

Portland Cement Paint.—A Portland cement paint has been used on the High St. viaduct in Columbus, Ohio, with good results. The viaduct was exposed to the fumes and blasts from locomotives, so that an ordinary paint did not last more than six months even on the least exposed portions. The method of mixing and applying the paint is described in Engineering News, April 24th and June 5th, 1902, as follows: "The surface of the metal was thoroughly cleaned with wire brushes and files—the bridge had been cleaned with a sand blast the previous year. A thick coat of Japan drier was then applied and before it had time to dry a coating was applied as follows: Apply with a trowel to the minimum thickness of $\frac{1}{16}$ in. and a maximum thickness of $\frac{1}{8}$ in. (in extreme cases $\frac{1}{4}$ in.) a mixture of 32 lb. Portland cement, 12 lb. dry finely ground lead, 4 to 6 lb. boiled linseed oil, 2 to 3 lb. Japan drier." After a period of about two years the coating was in almost perfect condition and the metal under the coating was as clean as when painted. The cost of the coating including the hand cleaning, materials and labor was 8 cts. per sq. ft.

INSTRUCTIONS FOR THE MILL INSPECTION OF STRUCTURAL STEEL.*

(1) Study the contract and specifications and secure such information concerning the proposed structure as will permit a full understanding of the use to be made of the various items of the order.

(2) Secure copies of the mill orders, shipping directions and other information concerning the material to be inspected.

(3) Attend promptly when notified of the rolling of material and so conduct the inspection and tests as not to interfere unnecessarily with the operations of the mill.

(4) Have the test specimens prepared and properly stamped with the melt numbers by the manufacturer. Observe the selection and stamping of specimens and verify the melt numbers when practicable.

(5) Attend and supervise the making of tensile, bending and drifting tests. Make sure that the testing machines are properly handled and that the specified speed of pulling is not exceeded. Note the behavior of the metal and check and record the results of the tests.

(6) Select the bars or other members for full-size tests as specified. Supervise such tests and check and record their results.

(7) Secure from the manufacturer records of the chemical analyses of the melts and accept only those in which the specified contents of impurities are not exceeded.

(8) Secure pieces of the test ingots and test specimens and have check analyses made outside of the manufacturers' laboratory when the analyses furnished by the manufacturer are erratic or for any other reason appear to be incorrect.

(9) Examine each piece of finished material for surface defects before shipment, requiring the material to be handled in a manner that will permit the examination to be thorough and complete. This inspection should detect evidence of excessive gagging or other injury due to cold straightening.

(10) Report promptly the shipment of any material from the mill, whose surface inspection has been waived. Such material should be examined by the shop inspector.

(11) Verify the section of all material by measurement and by weight.

(12) Study the operations of the plant and become familiar with the various processes of manufacture.

Cultivate the acquaintance of the mill employees and become familiar with their work so as to have direct knowledge of the mill practice and determine as well as the circumstances permit the correctness of the mill practice in so far as it is covered by the specifications.

(13) Record all tests and analyses on the forms provided.

(14) Keep informed as to the progress of the work in the shop and endeavor to secure the shipment of material at such times and in such order as to avoid delay in the fabrication.

(15) Secure copies of the shipping lists and compare them with the orders and make regular statements of the material that has been rolled and shipped.

(16) Make reports weekly or as may be directed, submitting complete records of tests, analyses and shipments and such other information as may be required.

* American Railway Engineering Association, Adopted, Vol. 14, 1913.

INSTRUCTIONS FOR THE INSPECTION OF THE FABRICATION OF STEEL BRIDGES.*

(1) Acquire a full knowledge of the conditions of the contract, such as the time of delivery, the railway company's actual need of the work, the desired order of shipment, and any special features in connection with delivery such as the position of the girders or truss members on cars at the bridge site.

(2) Study in advance the plans and specifications and see that all provisions thereof are complied with. These instructions are not be construed as altering the specifications in any way.

Check every finished member against the drawings for its general dimensions and for the section of each piece of material forming a component part of the member.

(3) Endeavor to maintain pleasant relations with foremen and the workmen and by fairness, decisiveness and good sense interest them in the successful completion of the work.

(4) Attend constantly to the work, making inspection during the progress of the work in the shop, striving to keep up with the output in order that errors may be corrected before the work leaves the shop.

Attend the weighing of material whenever practicable, especially that purchased on weight basis. Check the accuracy of the scales with test weights or by other sufficient means.

Conduct the inspection so as not to interfere unnecessarily with the routine operations of the shop.

(5) When unusual circumstances require an explanation of the plans or some variation from the specified procedure, take the necessary action promptly.

(6) Study the field connections, paying particular attention to clearances and making notations on the drawings so that they may be checked rapidly.

(7) Check all bevels and field rivet holes.

(8) Give careful attention to the quality of the workmanship, the condition of the plain material, accuracy of punching, care in assembling, alignment of rivets, tightness of rivets, a - curacy of finishing of machined joints, painting and general finish.

(9) Make sure that reamed holes are truly cylindrical and that drillings are not allowed to remain between assembled parts.

(10) Watch for bends, kinks, and twists in the finished members and make certain that when leaving the shop they are in proper condition for erection.

(11) Make sure that the webs of girders do not project beyond the flange angles and that the depth of web below the flange angles complies with the specification.

(12) Allow only the material rolled and accepted for the work to be used therein.

(13) Have the fabricated material shipped in the correct order for erection and in accordance with instructions, as far as practicable.

(14) Measure the width of each column and the lengths of all girders between columns when they are to be placed consecutively in a long row so as to insure that the columns and girders will not " build out " in erection, so as to exceed the calculated length.

(15) Check " rights " and " lefts " and make sure that the proper number of each is shipped.

(16) Check base plates of girders before riveting and make sure that the camber is not reversed.

(17) Check the space provided for driving field rivets, allowing sufficient space for the penumatic riveter.

(18) Examine field connections after riveting to insure proper fitting and ease of erection.

(19) Make sure that shop splices are properly fitted and that matched and milled surfaces to transmit bearing are in close contact during riveting as specified.

(20) Examine and measure bored pinholes carefully to insure proper dimensions and spacing and smoothness of finish.

(21) Measure the spacing center to center of the end connections for sections of I-beam floors or any similar construction in which the calculated spacing is liable to be exceeded because of the tendency of such work to " grow " as it is assembled.

(22) Make sure that stringers connecting to floorbeams beneath the flange have sufficient clearance to care for their possible over-run in depth.

(23) Have the assembling of trusses and girder spans required by the specifications carefully done and in any case insure the accuracy of field connections. If a large number of duplicate parts are to be made, the number of parts to be assembled should be governed by the workmanship. If errors are found, a sufficient number of parts should be assembled to make it reasonably certain that such errors have been eliminated.

Have through girder spans with I-beam floors partially assembled and at least one bracket bolted in its final position.

* American Railway Engineering Association, Adopted, Vol. 14, 1913, and Vol. 15, 1914.

Have at least one upper and lower shoe of each kind assembled and make sure that there is no interference.

(24) Make sure that iron templets used for reaming are properly set and held to line.

(25) Secure match-marking diagrams for work which has been assembled and reamed and make sure that the match marks are plainly visible.

(26) Have proper camber blocking used in assembling trusses and secure the desired camber before the reaming is done.

(27) Require that all treads and supports for the drums of draw spans be carefully leveled with an instrument.

(28) Study carefully the machine details and discriminate between those dimensions which must be exact and those in which slight variations are permissible.

Determine in advance the desired accuracy of driving fits for bolts or keys and similar parts and make sure that such accuracy is attained.

(29) Examine castings carefully for blowholes and other imperfections and discriminate between such defects as are unimportant and those which render the castings unfit for use.

(30) Make sure that bushings, collars and similar parts are held securely in place.

(31) Make sure that all drum wheels, expansion rollers, turntable rollers and similar parts are exact in size, so as to carry equally the loads which may be placed upon them.

(32) Ascertain in advance that the paint provided complies with specifications. Watch carefully the painting directions and make sure that paint is properly applied and only where intended.

(33) Verify all shop marks and make sure that they are legible as well as correct.

(34) Have important members so loaded as to be headed in the right direction upon arrival at the site of the work.

(35) Try a few countersunk head bolts in the holes where they are to be used to insure a proper fit.

(36) Make sure that small pieces are bolted in place for shipment as shown on the plans and that other small parts are properly boxed or otherwise secured against loss.

(37) Make sure that rivets, tie rods, anchor bolts and miscellaneous parts are shipped so as to avoid delay in erection.

(38) Examine the field rivets to insure that they are free from fins or other defects.

(39) Exercise special care in the examination of all movable structures and particularly their moving parts.

(40) Make reports weekly or as directed, exhibiting carefully and concisely the actual conditions.

(41) Observe carefully and report such unusual difficulties as may be encountered and the means adopted in overcoming them, and endeavor by a study of the details or other means to make recommendations which will prevent their recurrence in future work.

MISCELLANEOUS METALS.—The physical properties of the following metals depend upon whether they are cast, rolled, or drawn, and upon the details of manufacture, and the values given are therefore approximate.

Aluminum has a specific gravity of 2.58 to 2.7. The ultimate tensile strength per sq. in. is about 15,000 lb. for cast, 24,000 lb. for sheet, and 30,000 to 65,000 lb. for aluminum wire. The elastic limit is about $\frac{1}{3}$ the ultimate strength. The modulus of elasticity is about 11,000,000 lb. per sq. in. Aluminum is used in engineering construction principally in the form of an alloy.

Copper has a specific gravity of 8.6 to 8.9. The ultimate tensile strength varies from 36,000 to 40,000 lb. per sq. in. for soft copper wire with an elongation in 10 in. of 35 to 20 per cent; to 49,000 to 67,000 lb. per sq. in. for hard-drawn copper wire with an elongation varying from 3.75 per cent in 10 in., to an elongation of 0.85 per cent in 60 in. Copper is also used in an alloy with other metals.

Zinc, or spelter, has a specific gravity of about 7.00. The ultimate tensile strength per sq. in. varies from 3000 to 8000 lb. It is used for galvanizing and for making alloys.

Nickel has a specific gravity of about 8.8. Nickel is used principally in alloys.

Tin has a specific gravity of about 7.35. Tin is used as a covering for iron and steel sheets and in alloys.

Lead has a specific gravity of about 11.4. Lead is very plastic and flows easily under stress.

ALLOYS.—An alloy is a combination of two or more metals made by mixing them when in a molten condition. Alloys are commonly mechanical mixtures; although some have a slight chemical union. The properties of alloys depend not only upon the ingredients, but upon the method and

details of manufacture. It is impossible to predict the properties of an alloy from the properties of the metals forming it. Many alloys are sold under trade names in which the properties depend both on the proportions of the ingredients and upon the details of manufacture. The most important alloys used by the structural engineer are as follows:

Brass is an alloy of copper and zinc in which the copper varies from 60 to 89 per cent, and the zinc from 40 to 11 per cent. A small amount of tin is sometimes added to make the brass more easily worked. The tensile strength of brass is greatest (about 50,000 lb. per sq. in.) when the composition is about 62 per cent copper and 38 per cent zinc; and the ductility and malleability are greatest when the composition is about 70 per cent copper and 30 per cent zinc. A widely used brass has $\frac{2}{3}$ copper and $\frac{1}{3}$ zinc.

Delta metal is brass with 1 to 2 per cent iron. The tensile strength of delta metal is about 45,000 lb. per sq. in.

Tobin bronze is brass with 1 to 2 per cent iron, and small amounts of lead and tin.

Bronzes are alloys of copper and tin or of copper, zinc and tin, and usually have small quantities of other metals. Bronzes having more than 24 per cent tin are too weak to be used. The tensile strength is greatest (23,000 lb. per sq. in.) when the composition is about 80 per cent copper and 20 per cent tin.

Phosphor bronze is an alloy of copper and tin containing $\frac{1}{2}$ to 1 per cent phosphorus. It makes excellent castings and is very hard. The ultimate tensile strength varies from 50,000 to 100,000 lb. per sq. in.

Aluminum bronze is an alloy having 5 to 10 per cent aluminum and 95 to 80 per cent copper. The tensile strength varies from 75,000 to 100,000 lb. per sq. in.

Manganese-bronze as specified by the American Society for Testing Materials contains, copper 55 to 65 per cent, zinc 39 to 45 per cent, iron not over 2 per cent, tin not over 2 per cent. aluminum not over 0.5 per cent, manganese not over 0.5 per cent. The ultimate tensile strength of standard test pieces cut from manganese-bronze ingots shall not be less than 70,000 lb. per sq. in., with an elongation in 2 in. of not less than 20 per cent.

TIMBER.—For definitions of terms, standard defects, specifications and allowable stresses in timber, see Chapter VII.

STONE MASONRY.—For definitions of terms used in masonry construction and for specifications for different classes of stone masonry, see Chapter VI.

For the allowable pressure on masonry, see Table IV, Chapter V, and for the weight, specific gravity and crushing strength of masonry, see Table V, Chapter V; also see Table VIII, Chapter II. For an exhaustive treatise on brick and stone masonry see Baker's "Masonry Construction."

CONCRETE.—The average strengths of different mixtures of Portland cement concrete as given in Report of the Committee on Reinforced Concrete of the American Society of Civil Engineers, 1916, are given in Table II.

Specifications for concrete are given in Chapter V, and specifications for reinforced concrete are given in Chapter VI.

Working Stresses.—The following working stresses have been recommended by the American Railway Engineering Association for concrete that will develop an average compressive strength of at least 2000 lb. per sq. in. when tested in cylinders 8 in. in diameter and 16 in. long and 28 days old, under laboratory conditions of manufacture and storage, the mixture being of the same consistency as is used in the field.

	Lb. per sq. in.
Structural steel in tension	14,000
High carbon steel in tension	17,000
Steel in compression, 15 times the compressive stress in the surrounding concrete.	
Concrete in bearing where the surface is at least twice the loaded area	700
Concrete in direct compression, without reinforcement on lengths not exceeding 6 times the least width	450
Concrete in direct compression with not less than 1 per cent nor over 4 per cent longitudinal reinforcement on lengths not exceeding 12 times the least width	450

	Lb. per sq. in.
Concrete in compression, on extreme fiber in cross bending	750
Concrete in shear, uncombined with tension or compression in the concrete	120
Concrete in shear, where the shearing stress is used as a measure of the web stress	40
Note.—The limit of shearing stresses in the concrete, even when thoroughly reinforced for shear and diagonal tension, should not exceed	120
Bond for plain bars	80
Bond for drawn wire	40
Bond for deformed bars, depending on the form	100-150

ABSTRACT OF REPORT OF COMMITTEE ON CONCRETE AND REINFORCED CONCRETE OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS. The report was printed in Transactions of American Society of Civil Engineers, Vol. XLII, December, 1916.

The working stresses are given for static loads. Proper allowances are to be made for vibrations and impact. In selecting the proper working stress the designer should be guided by the working stresses used for other materials of construction, so that the entire structure may have the same degree of safety. The allowable stresses are given in terms of the ultimate compressive strength of concrete, obtained in testing concrete in cylinders 8 in. in diameter and 16 in. long, made of sluggish consistency, made and stored under laboratory conditions. The Committee recommends the following values for compressive strength of concrete to be used in design.

TABLE II.

COMPRESSIVE STRENGTH OF DIFFERENT MIXTURES OF CONCRETE, POUNDS PER SQUARE INCH.

Aggregate.	Proportions.*				
	1:3.	1:4½.	1:6.	1:7½.	1:9.
Granite, trap rock	3,300	2,800	2,200	1,800	1,400
Gravel, hard limestone, and hard sandstone	3,000	2,500	2,000	1,600	1,300
Soft limestone and sandstone	2,200	1,800	1,500	1,200	1,000
Cinders	800	700	600	500	400

ALLOWABLE STRESSES.

	Per Cent of Compressive Strength	Lb. per Sq. In.
Structural steel in tension		16,000
Concrete in compression where the surface is at least twice the loaded area	35	
Concrete for concentric compression on a plain concrete column or pier, the length of which does not exceed 4 diameters	22.5	
Compression on columns with longitudinal reinforcement only, length of the column shall not exceed 12 diameters	22.5	
(*a*) Columns with longitudinal reinforcement to the extent of not less than 1 per cent and not more than 4 per cent, and with lateral ties of not less than ¼ in. in diameter, 12 in. apart, nor more than 16 diameters of the longitudinal bar	22.5	
(*b*) Columns reinforced with not less than 1 per cent and not more than 4 per cent of longitudinal bars and with circular hoops or spirals not less than 1 per cent of the volume of the concrete and as hereinafter specified: a unit stress 55 per cent higher than given for (*a*) provided the ratio of unsupported length of column to diameter of the hooped core is not more than 10	34.875	

* Combined volume of fine and coarse aggregate measured separately.

The following limitations are placed on design of columns. Minimum size of columns 12 in. out to out. Longitudinal reinforcement to be assumed to carry its proportion of stress. Hoops or bands not assumed to carry stress. Hooping not to exceed 1 per cent of volume of column enclosed. Clear spacing of hooping not greater than one-sixth diameter enclosed column, preferably not greater than one-tenth and not more than $2\frac{1}{2}$ in. Ends of hooping must be united to develop full strength.

Compression on extreme fiber of a beam, calculated for constant modulus of elasticity (stresses adjacent to the supports of continuous beams may be 15 per cent higher) 32.5

Shear in beams with horizontal bars only and without web reinforcement 2

Shear in beams with vertical stirrups looped about longitudinal bars on tension side, and stirrups spaced horizontally not more than one-half the depth of the beam; or beams with longitudinal bars bent up at an angle not greater than 45° nor less than 20° with axis of beam, points of bending up spaced horizontally not more than three-quarters the depth of the beam 4.5

Shear in beams having a combination of bent bars and vertical stirrups looped about reinforcing bars in tension side of beam and spaced horizontally not more than one-half the depth of the beam 5

Shear in beams with web reinforcement (either vertical or inclined) securely attached to longitudinal bars in tension side of beam in such a way as to prevent slipping of bar past the stirrup, vertical stirrups spaced not more than one-half the depth of the beam, and inclined members spaced not more than three-quarters depth of beam 6

(The web reinforcement shall be proportioned for two-thirds the external vertical shear. The bent-up bars may be assumed as reducing the shearing stresses, but this reduction shall in no case be taken greater than $4\frac{1}{2}$ per cent of compressive strength of the concrete over the effective section of the beam. When calculated by the formula $f_v = V/(b \cdot j \cdot d)$, this would mean that shear f_v could not be greater than 90 lb. per sq. in. for 2,000 lb. concrete.)

The stresses in stirrups and inclined members when combined with bent-up bars are to be determined by finding the amount of the total shear that may be allowed by reason of the bent-up bars, and subtracting this shear from the total external vertical shear. Two-thirds of the remainder will be the shear to be carried by the stirrups.

The stresses in web reinforcement may be calculated by the following formulas:

Vertical web reinforcement

$$T = V' \cdot s'/j \cdot d \tag{75}$$

Bars bent up at angles between 20° and 45° with the horizontal and web members inclined at 45°

$$T = \frac{3}{4}\,\frac{V' \cdot s'}{j \cdot d} \tag{76}$$

Where V' = two-thirds of the total shear producing stress in the web reinforcement; T = total stress in member; s' = horizontal spacing of stirrups, and $j \cdot d$ = effective depth of beam.

Punching shear, only 6

Bond stress between concrete and plain reinforcing bars 4

Bond stress between concrete and drawn wire 2

Bond stress between concrete and deformed bars, not more than 5

The modulus of elasticity to be taken for the design as follows:

(*a*) One-fortieth that of steel, when the strength of the concrete is taken as not more than 800 lb. per sq. in.

(*b*) One-fifteenth that of steel where the strength of the concrete is taken as greater than 800 lb. per sq. in., and less than 2,200 lb. per sq. in., or less.

(*c*) One-twelfth that of steel where the strength of the concrete is taken greater than 2,200 lb. per sq. in. or less than 2,900 lb. per sq. in.

(*d*) One-tenth that of steel where the strength of concrete is taken as greater than 2,900 lb. per sq. in. In calculating deflection take one-eighth of the modulus of elasticity of steel.

Length of Beams and Columns.—The span length of beams and slabs simply supported should be taken as the distance center to center of supports, but need not be taken greater than the clear span plus the depth of beam or slab. For continuous or restrained beams built monolithically into the supports, the span length may be taken as the clear distance between faces of supports. Brackets should not be considered as reducing the clear span in the sense here intended, except that when brackets which make an angle of 45° or more with the axis of a restrained beam are built monolithically with the beam, the span may be measured from the section where the combined depth of the beam and the bracket is at least one-third more than the depth of the beam. Maximum negative moments are to be considered as existing at the end of the span as here defined. When the depth of a restrained beam is greater at its ends than at its mid-span and

the slope of the bottom of the beam at its ends makes an angle of not more than 15° with the direction of the axis of the beam at mid-span, the span length may be measured from face to face of supports.

The length of columns should be taken as the maximum unstayed length.

Design of T-beams.—In beam and slab construction an effective bond should be provided at the junction of beam and slab. When the principal reinforcement is parallel to the beam, transverse reinforcement should be used, extending over the beam and well into the slab.

The width of the slab shall not exceed one-fourth of the span length of the beam; and its overhanging width on each side of the web shall not exceed six times the thickness of the slab.

Floor-Slabs Supported along Four Sides.—Floor-slabs having the supports extending along the four sides should be designed and reinforced as continuous over the supports. For rectangular slabs in which the length is not greater than one and one-half times the width the portion of the total uniformly distributed load to be carried by the transverse reinforcement will be given by the formula $r = l/b - 0.5$, where l = length and b = width of slab. Two-thirds of the calculated moments shall be assumed as carried by the center half of the slab, and one-third by the outside quarters. The distribution of loads from slabs to the supporting beams shall be assumed as varying as the ordinates to a parabola with its vertex at the middle of the span.

Continuous Beams and Slabs.—When the beam or slab is continuous over its supports, reinforcement should be provided at points of negative moment. In computing bending moments for uniformly distributed loads the following rules are recommended:

(*a*) For floor-slabs, the bending moments at center and at support should be taken as $w \cdot l^2/12$ for both dead and live loads, where w represents the load per linear unit and l the span length.

(*b*) For beams, the bending moment at center and at support for interior spans should be taken as $w \cdot l^2/12$, and for end spans it should be taken as $w \cdot l^2/10$ for center and interior support, for both dead and live loads.

(*c*) In the case of beams and slabs continuous for two spans only, with their ends restrained, the bending moment both at the central support and near the middle of the span should be taken as $w \cdot l^2/10$.

(*d*) At the ends of continuous beams, the amount of negative moment which will be developed in the beam will depend on the condition of restraint or fixedness, and this will depend on the form of construction used. In the ordinary cases a moment of $w \cdot l^2/16$ may be taken; for small beams running into heavy columns this should be increased, but not to exceed $w \cdot l^2/12$.

For spans of unusual length, or for spans of materially unequal length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Even if the center of the span is designed for a greater bending moment than is called for by (*a*) or (*b*), the negative moment at the support should not be taken as less than the values there given.

Spacing of Bars.—Lateral spacing of parallel bars should not be less than three diameters center to center, nor two diameters from the side of the beam to the center of the bar. The clear spacing between two layers of bars should not be less than 1 in. The use of more than two layers is not recommended unless the layers are tied together by adequate metal connections, particularly at and near points where bars are bent up or down.

Reinforcement for Temperature.—Reinforcement not less than one-third of one per cent of a form that will develop a high bond resistance should be placed near the exposed surface and be well distributed.

STANDARD SPECIFICATIONS FOR PORTLAND CEMENT.

OF THE

AMERICAN SOCIETY FOR TESTING MATERIALS.

ADOPTED, 1920, (EFFECTIVE JAN. 1, 1921).

These specifications were approved March 31, 1922,

as "American Standard" by the

American Engineering Standards Committee.

1. **Definition.** Portland cement is the product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

I. CHEMICAL PROPERTIES.

2. **Chemical Limits.** The following limits shall not be exceeded:

Loss on ignition, per cent	4.00
Insoluble residue, per cent	0.85
Sulfuric anhydride (SO_3), per cent	2.00
Magnesia (MgO), per cent	5.00

II. PHYSICAL PROPERTIES.

3. **Specific Gravity.** The specific gravity of cement shall be not less than 3.10 (3.07 for white Portland cement). Should the test of cement as received fall below this requirement a second test may be made upon an ignited sample. The specific gravity test will not be made unless specifically ordered.

4. **Fineness.** The residue on a standard No. 200 sieve shall not exceed 22 per cent by weight.

5. **Soundness.** A pat of neat cement shall remain firm and hard, and show no signs of distortion, cracking, checking, or disintegration in the steam test for soundness.

6. **Time of Setting** The cement shall not develop initial set in less than 45 minutes when the Vicat needle is used or 60 minutes when the Gillmore needle is used. Final set shall be attained within 10 hours.

7. **Tensile Strength.** The average tensile strength in pounds per square inch of not less than three standard mortar briquettes composed of one part cement and three parts standard sand, by weight, shall be equal to or higher than the following:

Age at Test, days.	Storage of Briquettes.	Tensile Strength, lb. per sq. in.
7	1 day in moist air, 6 days in water	200
28	1 day in moist air, 27 days in water	300

8. The average tensile strength of standard mortar at 28 days shall be higher than the strength at 7 days.

III. PACKAGES, MARKING AND STORAGE.

9. **Packages and Marking.** The cement shall be delivered in suitable bags or barrels with the brand and name of the manufacturer plainly marked thereon, unless shipped in bulk. A bag shall contain 94 lb. net. A barrel shall contain 376 lb. net.

10. **Storage.** The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment, and in a suitable weather-tight building which will protect the cement from dampness.

IV. INSPECTION.

11. **Inspection.** Every facility shall be provided the purchaser for careful sampling and inspection at either the mill or at the site of the work, as may be specified by the purchaser. At least 10 days from the time of sampling shall be allowed for the completion of the 7-day test, and at least 31 days shall be allowed for the completion of the 28-day test. The cement shall be tested in accordance with the methods hereinafter prescribed. The 28-day test shall be waived only when specifically so ordered.

V. REJECTION.

12. **Rejection.** The cement may be rejected if it fails to meet any of the requirements of these specifications.

13. Cement shall not be rejected on account of failure to meet the fineness requirement if upon retest after drying at 100° C. for one hour it meets this requirement.

14. Cement failing to meet the test for soundness in steam may be accepted if it passes a retest using a new sample at any time within 28 days thereafter.

15. Packages varying more than 5 per cent from the specified weight may be rejected; and if the average weight of packages in any shipment, as shown by weighing 50 packages taken at random, is less than that specified, the entire shipment may be rejected.

References.—Camp and Francis' "The Making, Shaping and Treating Steel," published by Carnegie Steel Company, Pittsburgh, Pa.

CHAPTER XVI.

STRUCTURAL MECHANICS.

GENERAL NOMENCLATURE.—The following nomenclature will be used for all materials except reinforced concrete, for which a special notation is given.

A = area of cross section.
l = length or span.
L = length or span.
b = breadth of rectangular section.
d = depth of section; diameter of rivet.
t = thickness of plates, etc.
R = radius of circle.
D = diameter of circle.
h = height of wall.
c = distance from neutral axis to extreme fiber.
Δ = total deformation in length l, or maximum deflection of beams.
δ = unit deformation.
x = horizontal coordinate of elastic curve; variable.
y = vertical coordinate or deflection of elastic curve; variable.
e = eccentricity; efficiency.
I = moment of inertia.
I_c = polar moment of inertia.
J = product of inertia.
S = section modulus.
r = radius of gyration.
p = pitch of rivets.
P = concentrated load or total stress in a member.
f = unit fiber stress.
f_c = unit compressive fiber stress.
f_t = unit tensile fiber stress.
f_v = unit shearing fiber stress.
W = total uniformly distributed load; weight of a body.
w = uniformly distributed load per unit of length; load per unit of length at a distance unity from left end for a uniformly varying load; unit internal pressure.
R = reactions at supports.
M_x = moment at any section.
M = maximum moment.
V_x = total shear on any section.
V = maximum total shear.
E = modulus of elasticity.
G = shearing modulus of elasticity.
λ = Poisson's ratio.
$+$ = compressive stress.
$-$ = tensile stress.

REINFORCED CONCRETE NOMENCLATURE. Rectangular Beams, Reinforced for Tension Only.

f_s = tensile unit stress in steel, in pounds per square inch.
f_c = compressive unit stress in concrete, in pounds per square inch.
E_s = modulus of elasticity of steel, in pounds per square inch.
E_c = modulus of elasticity of concrete, in pounds per square inch.
n = elasticity ratio, $E_s \div E_c$.
M = bending moment, in inch-pounds.
M_s = moment of resistance of steel, in inch-pounds.
M_c = moment of resistance of concrete, in inch-pounds.
A = area of steel section, in square inches.
b = width of beam, in inches.
d = depth of beam to center of steel reinforcement, in inches.
k = ratio of depth of neutral axis to effective depth, d.
j = ratio of arm of resisting couple to depth, d.
p = steel ratio (not percentage), $A \div bd$.
C = total compressive stress in concrete, in pounds.
T = total tensile stress in steel, in pounds.

Tee Beams.

b = width of flange, in inches.
b' = width of stem, in inches.
t = thickness of flange, in inches.
p = steel ratio (not percentage), $A \div bd$.

See also " Rectangular Beams Reinforced for Tension Only."

Rectangular Beams, Reinforced for Compression.

A' = area of compressive steel, in square inches.
p' = steel ratio for compressive steel, $A' \div bd$.
f_s' = unit compressive stress in steel, in pounds per square inch.
C = total compressive stress in concrete, in pounds.
C' = total compressive stress in steel, in pounds.
T = total tensile stress in steel, in pounds.
d' = depth to center of compressive steel, in inches.
z = depth to resultant of compressive stresses, in inches.

See also " Rectangular Beams Reinforced for Tension Only."

Shear and Bond.

V = total shear in pounds.
f_v = unit shearing stress in concrete, in pounds per square inch.
f_u = unit bonding stress in concrete, in pounds per square inch.
Σ_0 = sum of the perimeters of the tension bars, in inches.
s = horizontal spacing of stirrups.
P = total stress carried by one stirrup.

Columns.

A = total net area, in square inches.
A_s = area of longitudinal steel, in square inches.
A_c = area of concrete, in square inches.
p = steel ratio, $A_s \div A$.
P = total axial load, in pounds.

DEFINITIONS.—The following definitions will be of service in a study of structural mechanics.

Forces.—Forces are concurrent when their lines of action meet in a point; non-concurrent when their lines of action do not meet in a point. Forces are coplanar when they lie in the same plane; or non-coplanar when they lie in different planes. Coplanar forces only will be here considered. A force is fully defined when its amount, its direction, and position are known.

Moment of Forces.—The moment of a force about a point is its tendency to produce rotation about that point, and is the product of the force and the perpendicular distance of the point from the line of action of the force.

Couple.—A couple is a pair of equal and opposite forces having different lines of action. The moment of a couple is equal to the product of one of the forces by the distance between the lines of action of the forces, or the arm of the couple.

Stress.—If a body be conceived to be divided into two parts by a plane traversing it in any direction, the force exerted between these two parts at the plane of division is an internal stress. Stress is force distributed over an area in such a way as to be in equilibrium. Stresses are measured in pounds, tons, etc.

Unit Stress is the measure of intensity of stress. The unit stress at any point is the number of units of stress acting on a unit of area at that point. Unit stresses are expressed in pounds per square inch, tons per square foot, etc.

Ultimate Stress.—Ultimate stress is the greatest stress which can be produced in a body before rupture occurs.

Tension is the name for the stress which tends to prevent the two adjoining parts of a body from being pulled apart when the body is acted upon by two forces acting away from each other.

Compression is the name of the stress which tends to keep two adjoining parts of a body from being pushed together under the influence of two forces acting toward each other.

Shear is the name of the stress which tends to keep two adjoining planes of a body from sliding on each other under the influence of two equal and parallel forces acting in opposite directions.

Axial Stresses.—When the external forces producing tension or compression act through the center of a gravity of the body the stresses are uniformly distributed over the area, and the stresses are axial stresses.

Simple Stress.—If P = the force producing tension, compression, or shear and A = the area over which the stress is distributed, then

$$f_t = P/A; \quad f_c = P/A; \quad f_v = P/A,$$

where f_t is tensile stress, f_c is compressive stress, and f_v is shearing stress.

Working Stress.—The working stress for any material is the unit stress that has been found by experiment to be safe to allow for that particular material to give a properly designed structure. The working stress for any particular structure depends upon the material of which the structure is built, the loads that the structure is to carry, the accuracy with which the loads and stresses have been calculated, the possible defects in the material, etc.

Factor of Safety.—The factor of safety is the number by which the ultimate stress must be divided to give the working stress.

Deformation or Strain is the change in the shape of a body caused by the action of an external force. Deformation or strain is measured in linear units. Deformation may be due to tension, elongation; due to compression, shortening; or due to shear, detrusion or slipping of one plane past another.

Elasticity.—Up to a certain stress in an elastic body it has been found by experiment that stress is proportional to strain. This principle is known as "Hooke's Law." The ability of a body to return to its original form after deformation is termed elasticity. If the stress in a body is carried beyond a certain limit the body does not return to its original form, but a permanent set occurs.

Elastic Limit.—The elastic limit of a material is the highest unit stress to which that material may be subjected and still return to its original shape when the stress is removed, and is the limit within which the stresses are directly proportional to the deformations.

Yield Point.—In testing materials a point is reached beyond the elastic limit where unit elongations increase very rapidly without any or with a very slight increase in unit stress. This point is indicated by the drop of the scale beam of the testing machine. In steel the yield point is from three to six thousand pounds per square inch above the elastic limit.

Modulus of Elasticity.—The modulus of elasticity of a material is the constant, which within the elastic limit expresses the ratio between the unit stress and unit strain or deformation. If E = modulus of elasticity, P = an axial force; A = cross sectional area of the bar, f = unit stress = P/A; Δ = deformation produced by P in a length l, and $\delta = \Delta/l$; then

$$E = (P/A)/(\Delta/l) \quad \text{or} \quad E = f/\delta.$$

The modulus of elasticity may be defined as that force, were Hooke's law applicable without limit, which would produce in a bar with a cross section of one square inch a deformation equal to its original length.

The modulus of elasticity of steel is very closely E = 30,000,000 lb. per sq. in.; the modulus of elasticity of timber is approximately E = 1,500,000 lb. per sq. in.; while the modulus of elasticity of concrete varies from E = 1,500,000 lb. per sq. in. to E = 3,000,000 lb. per sq. in. with an average value of E = 2,000,000 lb. per sq. in.

Shearing Modulus of Elasticity.—The shearing modulus of elasticity, also called the modulus of rigidity, is the modulus expressing the ratio between unit shearing stress and unit shearing strain. The value of shearing modulus of elasticity for steel is about $\frac{2}{5}$ of the value of E, or G = 12,000,000 lb. per sq. in.

Poisson's Ratio.—Direct stress produces a strain in its own direction and an opposite kind of strain in every direction perpendicular to its own. For example a bar under tensile stress extends longitudinally and contracts laterally. Poisson's ratio is the ratio of lateral strain to longitudinal strain, and is a constant below the elastic limit. For steel Poisson's ratio is $\frac{1}{4}$ to $\frac{1}{3}$, while for concrete it is from $\frac{1}{8}$ to $\frac{1}{10}$.

Rupture Strength.—In testing steel the cross sectional area rapidly decreases beyond the ultimate stress and if the rupture stress be divided by the original cross sectional area the unit stress at rupture will be less than the ultimate stress.

Ultimate Deformation.—The ultimate deformation is the total deformation in a prescribed length, commonly 8 inches, or 2 inches. It is usually expressed in per cent for a length of 8 inches, or of 2 inches.

Work or Resilience in a Bar.—The amount of work that can be stored up in a body under stress within the elastic limit is called resilience or "internal work." When the external force has been gradually applied all the work may be recovered when the force is removed.

From the law of conservation of energy the external work due to the force is equal to the resilience or internal work. If a load P is supported at the lower end of a bar without weight, having a length l and a cross sectional area A; then the external work will be $\frac{1}{2}P\cdot\Delta$, where Δ = the total deformation, and the internal work or resilience will be

$$K = \frac{P}{2}\left(\frac{P\cdot l}{A\cdot E}\right) = \frac{1}{2}\left(\frac{P^2}{A^2\cdot E}\right) A\cdot l = \frac{1}{2}\left(\frac{f^2}{E}\right) A\cdot l,$$

when f = elastic limit of the material then $\frac{1}{2}f^2/E$ is termed the *Modulus of Resilience.*

Stresses due to Sudden Loads.—In a bar acted on by a static load, P, gradually applied, the total resilience will be $K = \frac{1}{2}\Delta . P$. If the load P is suddenly applied we will have $K = \Delta . P$, from which it is seen that the stress produced by a sudden load is twice that produced by a load gradually applied.

Impact.—The stresses due to moving loads are greater than the stresses due to loads at rest. The increase in stress of the moving load over the load at rest is called impact. For a discussion of impact stresses in railway bridges see page 161, Chapter IV.

STRESSES IN BEAMS.—When a straight beam or bar is supported near the ends and carries loads or forces applied transverse to the length of the axis of the beam or bar, the axis of the member assumes a curve. The transverse loads or forces are carried by flexure, which is a combination of the three simple stresses of tension, compression and shear. For example, a simple beam resting horizontally on supports carries a concentrated load. The fibers on the lower or convex side of the beam will be elongated and are therefore in tension, while the fibers on the upper or concave side are shortened and are therefore in compression. Shear is taking place between each vertical plane of the beam and the plane adjoining between the load and each support. Since the longitudinal stresses in a simple beam vary from a maximum compression on the concave side to a maximum tension on the convex side, the stresses will pass through zero on some plane, called the neutral plane or axis. Also since the fibers on each side of the neutral axis carry different amounts of stress, they will lengthen or shorten different amounts, and there will therefore be horizontal shearing stresses as well as vertical shearing stresses.

Neutral Surface and Neutral Axis.—Under flexure a beam is curved, and the fibers on the concave side are in compression while the fibers on the convex side are in tension. The neutral surface is a surface on which the fibers have zero stress, and the neutral axis is the trace of this plane on any longitudinal section of the beam. In a simple horizontal beam carrying vertical loads the neutral axis passes through the center of gravity of the cross section of the beam, for a rectangular beam the neutral axis is at half the height of the beam. Where a beam carries loads that are not at right angles to the neutral axis of the beam, the beam is in equilibrium under flexure and direct stress, and the neutral axis or line of zero stress will not pass through the center of gravity of the cross section of the beam, and may fall entirely outside the beam. A bar carrying simple tension or compression may be considered as a beam in which the neutral axis is at an infinite distance from the center of gravity of the cross section of the beam.

Reactions.—For any structure to be in equilibrium, (1) the sum of the horizontal components of all forces acting on the beam must equal zero, (2) the sum of the vertical components of all forces acting on the beam must equal zero, and (3) the sum of the moments about any point of all forces acting on the beam must be equal to zero. Having the loads given the reactions can be calculated by applying the three conditions of equilibrium.

Vertical Shear.—The vertical shear in a beam is equal to the algebraic sum of the forces (reaction minus the loads) on the left of the section considered.

Bending Moment.—The bending moment at any section of a beam is equal to the algebraic sum of the moments of the reaction and the loads on the left of the section.

Relations between Shear and Bending Moment.—In a simple beam carrying vertical loads the shear is a maximum at the supports and passes through zero at some intermediate point in the beam. The bending moment is zero at the supports and is a maximum at some intermediate point in the beam. The shear is the algebraic sum of all the forces on the left of a section, while the bending moment may be defined as the algebraic sum of all the shearing stresses on the left of the section. The definite integral of the loads to the left of the section equals the shear at the section, and the definite integral of the shear to the left of the section is equal to the bending moment at the section. From the above it will be seen that maximum bending moment will come at the point of zero shear.

Formulas for Flexure.—Applying the conditions for static equilibrium to any cross section of a beam we have, (1) Sum of Tensile Stresses = Sum of Compressive Stresses; (2) Resisting Shear = Vertical Shear; (3) Resisting Moment = Bending Moment.

Resisting Shear.—If the shearing stresses are uniformly distributed the shearing stress will be

$$f_s = V/A. \qquad (1)$$

The shearing stresses are not uniformly distributed and for a rectangular beam $f_s = \frac{3}{2}V/A$, while in a circular beam $f_s = \frac{4}{3}V/A$.

Resisting Moment.—The bending moment at any section is resisted by the moment of the tensile and compressive stresses which act as a couple with an arm equal to the distance between the centroids of the tensile and compressive stresses. The moment of this internal couple is called the resisting moment. If f = the unit stress at any extreme fiber on the surface of the beam due to bending moment, c = distance from that fiber to the neutral axis, and M = the bending moment, or the resisting moment, then

$$M = \frac{f \cdot I}{c}, \quad \text{or} \quad f = \frac{M \cdot c}{I}, \tag{2}$$

where I = the moment of inertia of the cross section of the beam.

Moment of Inertia.—The moment of inertia of any area about any axis is equal to the sum of the products obtained by multiplying each differential area, dA, by z^2, the square of the distance of each elementary area from the axis, $I = \Sigma z^2 \cdot dA$. The moment of inertia of any section is a minimum when the axis passes through the center of gravity of the cross section.

Section Modulus.—In designing beams it is convenient to use the ratio $S = I/c$, so that $M = f \cdot S$, or $f = M/S$. The ratio S is known as the section modulus.

Tables of Moments of Inertia and Section Modulus.—Values of moment of inertia, I, and section modulus, S, for different sections are given on pages 548 to 551, inclusive. Values of moment of inertia and section modulus of structural shapes are given in Part II.

Deflection of Beams.—In a simple beam carrying vertical loads the upper fibers are shortened and the lower fibers are lengthened, while the fibers on the neutral axis are not changed in length but the neutral axis assumed the form of a curve. The differential equation of the elastic curve of a horizontal beam carrying vertical loads will be

$$\frac{d^2y}{dx^2} = \frac{M}{E \cdot I}. \tag{3}$$

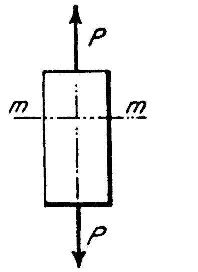
Substituting proper values of E, I and M, integrating twice and giving proper values to the constants of integration, the values y, or the deflection may be calculated for any point in the beam. The equation of the elastic curve of beams of various types are given on pages 649 to 665, inclusive.

The maximum bending moments and shears in beams due to moving concentrated loads are given on page 660.

The moments and shears in continuous beams are given on page 661, page 662 and page 663.

Formulas for stresses in reinforced concrete beams are given on page 664, and stresses in columns, safe working stresses, and safe loads on slabs are given on page 665

1. AXIAL TENSION.



Unit tension on m-m,

$$f_t = \frac{P}{A}, \qquad (a)$$

Total tension on m-m,

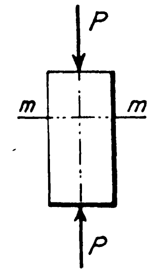
$$P = f_t A, \qquad (b)$$

Area for given stress,

$$A = \frac{P}{f_t}, \qquad (c)$$

where A = area section m-m

2. AXIAL COMPRESSION.



Unit compression on m-m,

$$f_c = \frac{P}{A}, \qquad (a)$$

Total compression on m-m,

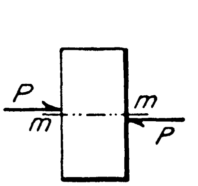
$$P = f_c A \qquad (b)$$

Area for given stress,

$$A = \frac{P}{f_c} \qquad (c)$$

where A = area of section m-m.

3. SIMPLE SHEAR.



Unit shear on m-m,

$$f_v = \frac{P}{A}, \qquad (a)$$

Total shear on m-m,

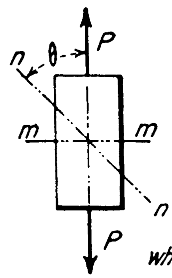
$$P = f_v A, \qquad (b)$$

Area for given stress,

$$A = \frac{P}{f_v}, \qquad (c)$$

where A = area section m-m

4. DIAGONAL STRESSES: TENSILE FORCE.



Unit shear on n-n,

$$f = \frac{1}{2}\frac{P}{A}\sin 2\theta = \frac{1}{2} f_t \sin 2\theta \qquad (a)$$

Unit tension on n-n,

$$f = \frac{P}{A}\sin^2\theta = f_t \sin^2\theta \qquad (b)$$

Max. unit shear on n-n,

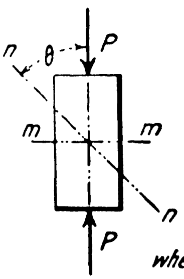
$$f = \frac{1}{2} f_t;\ \theta = 45°; \qquad (c)$$

Max. unit tension on n-n,

$$f = f_t;\ \theta = 90°; \qquad (d)$$

where $f_t = \frac{P}{A}$, A = area of section m-m.

5. DIAGONAL STRESSES: COMPRESSIVE FORCE.



Unit shear on n-n,

$$f = \frac{1}{2}\frac{P}{A}\sin 2\theta = \frac{1}{2} f_c \sin 2\theta; \qquad (a)$$

Unit compression on n-n,

$$f = \frac{P}{A}\sin^2\theta = f_c \sin^2\theta \qquad (b)$$

Max. unit shear on n-n,

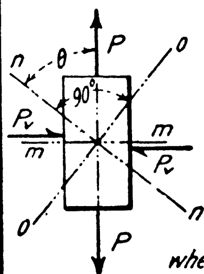
$$f = \frac{1}{2} f_c;\ \theta = 45°; \qquad (c)$$

Max. unit compression on n-n,

$$f = f_c;\ \theta = 90°; \qquad (d)$$

where $f_c = \frac{P}{A}$, A = area section m-m.

6. DIAGONAL STRESSES: TENSILE & SHEARING FORCES.



Max. unit shear on n-n;

$$f = \left[f_v^2 + \frac{1}{4} f_t^2\right]^{\frac{1}{2}};\ \tan 2\theta = +\frac{f_t}{2f_v}; \qquad (a)$$

Max. unit tension on n-n,

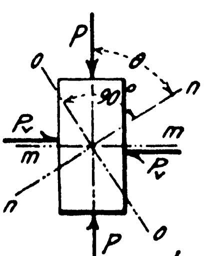
$$f = \frac{1}{2} f_t + \left[f_v^2 + \frac{f_t^2}{4}\right]^{\frac{1}{2}};\ \cot 2\theta = -\frac{f_t}{2f_v}, \qquad (b)$$

Max unit compression on o-o;

$$f = \frac{1}{2} f_t - \left[f_v^2 + \frac{f_t^2}{4}\right]^{\frac{1}{2}};\ \cot 2\theta = -\frac{f_t}{2f_v}; \qquad (c)$$

where $f_t = \frac{P}{A}$, $f_v = \frac{P_v}{A}$, A = area sec. m-m

7. DIAGONAL STRESSES: COMPRESSIVE & SHEARING FORCES.



Max. unit shear on n-n;

$$f = \left[f_v^2 + \frac{f_c^2}{4}\right]^{\frac{1}{2}};\ \tan 2\theta = +\frac{f_c}{2f_v}; \qquad (a)$$

Max. unit compression on n-n,

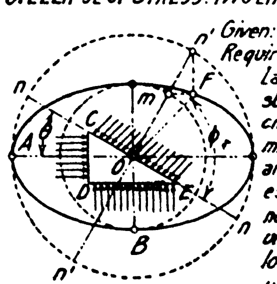
$$f = \frac{f_c}{2} + \left[f_v^2 + \frac{f_c^2}{4}\right]^{\frac{1}{2}};\ \cot 2\theta = -\frac{f_c}{2f_v}; \qquad (b)$$

Max unit tension on n-n,

$$f = \frac{f_c}{2} - \left[f_v^2 + \frac{f_c^2}{4}\right]^{\frac{1}{2}};\ \cot 2\theta = -\frac{f_c}{2f_v}; \qquad (c)$$

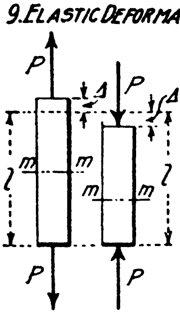
where $f_c = \frac{P}{A}$; $f_v = \frac{P_v}{A}$; A = area sec. m-m.

8. ELLIPSE OF STRESS: TWO LIKE STRESSES.



Given: Unit stresses on CD & DE. Required Unit stress on CE. Lay off AO and BO = unit stresses on CD & DE. Draw circles, n'O normal to CE, mF and n'F parallel to AO and BO. Then FO = unit stress on CE. FO sin φ = unit normal stress. FO cos φ unit shear. Ellipse is locus of F for all values of θ.

9. ELASTIC DEFORMATION: TENSION AND COMPRESSION.



Modulus of Elasticity,

$$E=\frac{f}{\delta}=\frac{P/A}{\Delta/l}=\frac{Pl}{A\Delta} \qquad (a)$$

Total deformation,

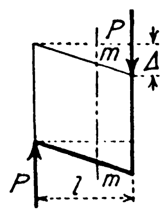
$$\Delta=\frac{f}{E}l=\frac{Pl}{AE} \qquad (b)$$

Unit deformation,

$$\delta=\frac{f}{E}=\frac{P}{AE} \qquad (c)$$

where A = area section m-m.

10. ELASTIC DEFORMATION: SHEAR.



Modulus of Elasticity,

$$G=\frac{f}{\delta}=\frac{P/A}{\Delta/l}=\frac{Pl}{A\Delta} \qquad (a)$$

Total deformation,

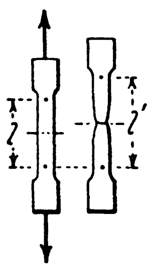
$$\Delta=\frac{f}{G}l=\frac{Pl}{AG} \qquad (b)$$

Unit deformation,

$$\delta=\frac{f}{G}=\frac{P}{AG} \qquad (c)$$

where A = area section m-m.

11. ULTIMATE DEFORMATION:



Percent elongation,

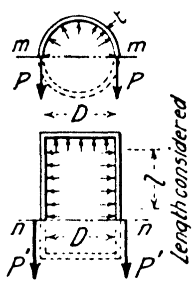
$$\frac{l'-l}{l}\cdot 100 \qquad (a)$$

Percent reduction of area,

$$\frac{A-A'}{A}\cdot 100 \qquad (b)$$

l = Original length.
l' = Length at failure.
A = Original section area.
A' = Area ruptured section.

12. THIN PIPES AND CYLINDERS: INTERNAL PRESSURE.



Longitudinal rupture, sec. m-m,

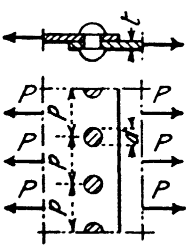
$$P=\frac{wDl}{2};\; f=\frac{wD}{2t} \qquad (a)$$

Transverse rupture, sec. n-n.

$$P'=\frac{w\pi D^2}{4};\; f=\frac{wD}{4t} \qquad (b)$$

w = unit internal pressure.
Both longitudinal and transverse stresses are independent of the form of the ends.

13. STRESSES IN SINGLE RIVETED LAP JOINTS.



Unit tension on plate,

$$f_t=P\div(p-d)\cdot t \qquad (a)$$

Unit compression on rivet,

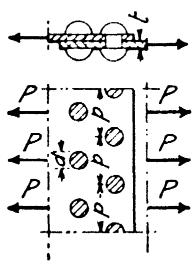
$$f_c=P\div td \qquad (b)$$

Unit shear on rivet,

$$f_v=P\div\tfrac{1}{4}\pi d^2 \qquad (c)$$

For longitudinal joints in pipes or cylinders $P=\frac{1}{2}wDp$; (d)
D = diam. pipe or cylinder.

14. STRESSES IN DOUBLE RIVETED LAP JOINTS.



Unit tension on plate,

$$f_t=P\div(p-d)\cdot t \qquad (a)$$

Unit compression on rivet,

$$f_c=P\div 2td \qquad (b)$$

Unit shear on rivet,

$$f_v=P\div\tfrac{1}{2}\pi d^2 \qquad (c)$$

For longitudinal joints in pipe or cylinders $P=\frac{1}{2}wDp$, (d)
D = diam. of pipe or cylinder.

15. DESIGN OF SINGLE RIVETED LAP JOINTS.

See figure above. For Butt Joints see Chapt. XVII

Most efficient joint for cylinders and pipe,

$$e=\frac{f_c}{f_t+f_c};\; t=\frac{wD}{2f_te};\; d=\frac{4f_c}{\pi f_v}t;\; p=\left[1+\frac{f_c}{f_t}\right]d;$$

(a) (b) (c) (d)

Most efficient joint for given thickness plate;

$$d=\frac{4f_c}{\pi f_v}t;\quad p=\left[1+\frac{f_c}{f_t}\right]d;\quad e=\frac{p-d}{p};$$

(e) (f) (g)

For joints with more than two rows of rivets see Chapt. XVII.

16. DESIGN OF DOUBLE RIVETED LAP JOINTS.

See figure above.

Most efficient joint for cylinders and pipe,

$$e=\frac{2f_c}{f_t+2f_c};\; t=\frac{wD}{2f_te};\; d=\frac{4f_c}{\pi f_v}t;\; p=\left[1+\frac{2f_c}{f_v}\right]d;$$

(a) (b) (c) (d)

Most efficient joint for given thickness plate,

$$d=\frac{4f_c}{\pi f_v}t;\quad p=\left[1+\frac{2f_c}{f_t}\right]d;\quad e=\frac{p-d}{p};$$

(e) (f) (g)

For joints with more than two rows of rivets See Chapt. XVII

17. FLEXURE FORMULA.

Fiber stress due to a given moment in a given beam,

$$f=\frac{Mc}{I} \quad (a)$$

Moment to cause a given fiber stress in a given beam,

$$M=\frac{fI}{c} \quad (b)$$

Section modulus for given moment and fiber stress,

$$S=\frac{I}{c}=\frac{M}{f} \quad (c)$$

Moment of inertia for given moment, fiber stress and distance to extreme fiber,

$$I=\frac{Mc}{f} \quad (d)$$

18. ELASTIC DEFLECTION OF BEAMS.

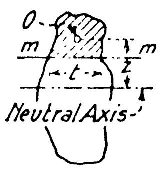
Differential equation from which equation of elastic curve is found,

$$EI\frac{\delta^2 y}{\delta x^2}=M_x \quad (a)$$

To determine elastic curve, when I and E are constant, integrate twice determining constants of integration by substituting known values of slope and deflection and corresponding values of x.

The equation of curve changes at every concentrated load but is same throughout for uniform load or for uniformly varying load.

19. SHEARING STRESSES IN BEAMS.



O is centroid of shaded area

Average unit shearing stress,

$$f_v=\frac{V}{A}, \quad (a)$$

Unit horizontal shearing stress, (longitudinal shear)

$$f_v=\frac{V}{It}\mathfrak{m}, \quad (b)$$

$\mathfrak{m}$ = static moment of area, above section considered, about neutral axis. For horizontal shear at m-m, $\mathfrak{m}$ = area of shaded portion multiplied by z, the distance to its centroid. The max. unit horizontal shear will occur at the neutral axis.

The max. unit horizontal shear for a rectangular beam = $\frac{3}{2}$ average unit shear, for circular section, $\frac{4}{3}$, and for an I-beam may be as much as $2\frac{1}{2}$ times average unit shear.

For rolled or built I-beams the max. unit horizontal shear very nearly equals the total vertical shear divided by area of web.

20. COLUMN FORMULAS: AXIAL LOADS.

Straight Line Formula,

$$\frac{P}{A}=\alpha-\beta\frac{l}{r} \quad (a)$$

For constants α and β see Table IX page 104.

Rankine's (Gordon's) Formula,

$$\frac{P}{A}=\frac{\alpha'}{1+\beta'\frac{l^2}{r^2}} \quad (b)$$

For constants α' and β' see Table IX page 80.

Euler's Formula,

$$\frac{P}{A}=\alpha'' E\frac{r^2}{l^2} \quad (c)$$

According to Merriman α'' has the following values;

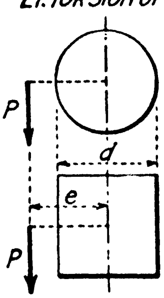
Both ends hinged, $\alpha''=\pi^2$

One end fixed and one hinged, $\alpha''=2\frac{1}{4}\pi^2$

Both ends fixed, $\alpha''=4\pi^2$

In Euler's Formula P = ultimate strength.

21. TORSION OF SHAFTS.



Solid round shafts,

$$Pe=\frac{1}{16}\pi d^3 f \quad (a)$$

$$f=321{,}000\frac{H}{Nd^3} \quad (b)$$

$$d=68.5\left[\frac{H}{Nf}\right]^{\frac{1}{3}} \quad (c)$$

Solid square shafts,

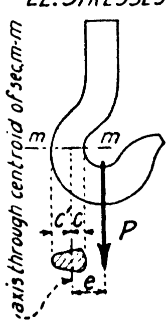
$$Pe=\frac{2}{9}d^3 f \text{ (approx.)} \quad (d)$$

$$f=284{,}000\frac{H}{Nd^3} \text{ ''} \quad (e)$$

$$d=65.7\left[\frac{H}{Nf}\right]^{\frac{1}{3}} \text{ ''} \quad (f)$$

H = horse power.
N = rev. per minute.

22. STRESSES IN HOOKS: Approximate Solution.



Maximum tension,

$$f=\frac{P}{A}+\frac{Pec}{I} \quad (a)$$

where A = area of section m-m, e = distance from line of action of load, P, to centroid of mm, c = distance from centroid to extreme fiber on tension side, I = moment of inertia of section m-m about axis through centroid.

For exact solution see "Slocum and Hancock," p191.

23 PLATE GIRDERS: See also Chapter XVII

(1) Moment all carried by flanges,
$M = A_f' f h$ (a)

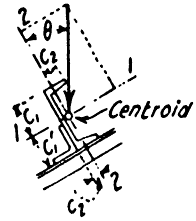
(2) One-eighth area of web available as flange area. $M = (A_f' + \frac{1}{8}A_w) f h$ (b)

(3) Moment of inertia of net section,
$M = \frac{fI'}{c}$ (c)

(4) Moment of inertia of gross section,
$M = \frac{fI}{c}$ (d)

A_f and A_w = net area of one flange and gross area of web, I and I' = moment of inertia of gross and of net section, h = dist. ℄ to ℄ of flanges.

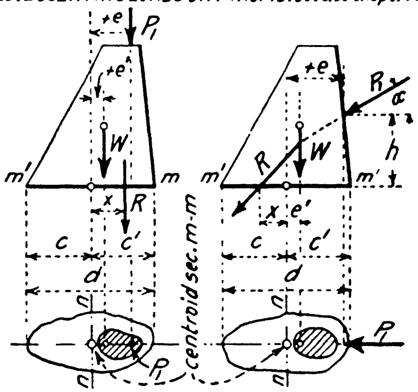
24 UNSYMMETRICAL LOADS ON BEAMS. Approximate Solution.



M = max moment for vertical loads
I_1 = moment of inertia, axis 1-1
I_2 = moment of inertia, axis 2-2

Max compressive fiber stress,
$$f_c = M\left(\frac{c_1\cos\theta}{I_1} + \frac{c_2\sin\theta}{I_2}\right), \quad (a)$$
Max tensile fiber stress;
$$f_t = M\left(\frac{c_1'\cos\theta}{I_1} + \frac{c_2'\sin\theta}{I_2}\right), \quad (b)$$

25. ECCENTRIC LOADS ON PRISMS: See also Chapt. V.



$P = P_1 + W$
$M = P_1 e + We'$

$P = P_1 \sin\alpha + W$
$M = P_1 \sin\alpha\, e - P_1\cos\alpha\, h + We'$

Stress at m, $f = \frac{P}{A} + \frac{Mc}{I_n}$; Stress at m', $f = \frac{P}{A} - \frac{Mc}{I_n}$;

I_n = moment of inertia of section m-m' about axis n-n.
A = area of section m-m'.
Line of action of resultant, $x = M \div P$;
If there is tension at m' and section will not take it, the stress at m' = 0 and at $m = \frac{2}{3}P\left(\frac{d}{2} - x\right)$ for rectang. sect.

26. FLEXURE AND DIRECT STRESS.

Flexure and compression, $f = \frac{P}{A} \pm \frac{Mc}{I - (Pl^2 \div kE)}$; (a)

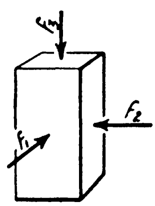
Flexure and tension, $f = \frac{P}{A} \pm \frac{Mc}{I + (Pl^2 \div kE)}$, (b)

$k = 10$ for both ends hinged, 24 for one end hinged and one fixed, 32 for both ends fixed.

Approximate formula, $f = \frac{P}{A} \pm \frac{Mc}{I}$; (c)

for direct stress either tension or compression. M may be due to weight of member or to external load.

27. TRUE STRESS.



$f_1, f_2, \& f_3$ = apparent unit stresses.
$t_1, t_2, \& t_3$ = true unit stresses.
$t_1 = f_1 - \lambda f_2 - \lambda f_3$; (a)
$t_2 = f_2 - \lambda f_1 - \lambda f_3$; (b)
$t_3 = f_3 - \lambda f_1 - \lambda f_2$, (c)
If any stress is tension change its sign in above formulas.
$\lambda = \frac{1}{3}$ for steel and wrought iron.
$\lambda = \frac{1}{4}$ for cast iron.
$\lambda = \frac{1}{10}$ for concrete.

λ = Poisson's Ratio.

28. CYLINDRICAL ROLLERS.

Unit Stress for given load and roller, $f = \left[\frac{9W^2E}{8L^2D^2}\right]^{\frac{1}{3}}$ (a)

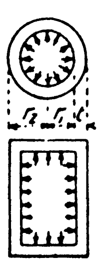
Length for given load, diam. and unit stress, $L = \frac{3W}{2fD}\left[\frac{E}{2f}\right]^{\frac{1}{2}}$; (b)

Total load for given roller and unit stress. $W = \frac{2}{3}LDf\left[\frac{2f}{E}\right]^{\frac{1}{2}}$; (c)

Load per unit length for given roller and unit stress. $w = \frac{2}{3}Df\left[\frac{2f}{E}\right]^{\frac{1}{2}}$ (d)

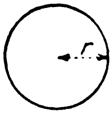
D = diam. of roller. L = length of roller,
E = modulus of elasticity.

29. THICK PIPES AND CYLINDERS: Internal Pressure.



Maximum unit tension,
$$f_t = w\frac{r_2^2 + r_1^2}{r_2^2 - r_1^2}; \quad (a)$$
Maximum unit compression,
$$f_c = w \quad (b)$$
Thickness for given pressure, unit tension and internal radius
$$t = r_1\left[\left(\frac{f_t + w}{f_t - w}\right)^{\frac{1}{2}} - 1\right] \quad (c)$$
w = unit internal pressure.

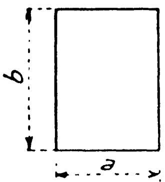
30. STRESSES IN FLAT PLATES·UNIFORM LOAD.



Circular Plate;

Circumference fixed,
$$f=\frac{45wr^2}{64t^2}.$$

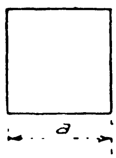
Circumference supported,
$$f=\frac{117wr^2}{128t^2}$$



Rectangular Plate,

Circumference fixed,
$$f=\frac{b^4\cdot w\cdot b^2}{2(a^4+b^4)t^2},$$

Circumference supported,
Unit stress is about $\frac{3}{2}$ that for circumference fixed.



Square Plates,

Circumference fixed,
$$f=\frac{wa^2}{4t^2};$$

Circumference supported,
Unit stress is about $\frac{3}{2}$ that for circumference fixed.

See Chapter VIII, p 395 and Table 135.

31. WORK OR RESILIENCE.

BARS.

Work done in stressing a bar below elastic limit. From 0 to P, or 0 to f,
$$K=\frac{1}{2}P\Delta=\frac{1}{2}f\delta Al=\frac{1}{2}\left(\frac{f^2}{E}\right)Al \qquad (a)$$

From P_1 to P_2 or f_1 to f_2,
$$K=\frac{1}{2}P_2\Delta_2-\frac{1}{2}P_1\Delta_1=\frac{1}{2}(f_2\delta_2-f_1\delta_1)Al; \qquad (b)$$
$$=\frac{1}{2}\left(\frac{f_2^2-f_1^2}{E}\right)Al,$$

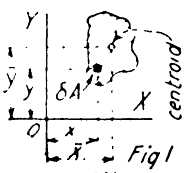
BEAMS.

Deflection under one load
$$y=\frac{1}{P}\int\frac{M_x^2}{EI}\delta x \qquad K=\int\frac{M_x^2}{2EI}\delta x, \qquad (c)$$

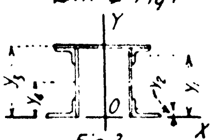
Deflection at any point,
$$y=\int\frac{M_xM'\delta x}{EI}; \qquad (d)$$

where M_x = moment at any point due to given loading and M' = moment at any point due to a unit load placed at the point at which the deflection is required.

32. CENTROID (CENTER OF GRAVITY).

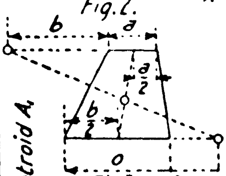


General Formulas;
$$\bar{x}=\frac{\int x\delta A}{\int\delta A}=\frac{\int x\delta A}{A}; \qquad (a)$$
$$\bar{y}=\frac{\int y\delta A}{\int\delta A}=\frac{\int y\delta A}{A}; \qquad (b)$$



Structural sections can be divided into finite elements the properties of which are known. Then (a) and (b) become
$$\bar{x}=\frac{\Sigma x\Delta A}{A}=\frac{\Sigma\mathfrak{M}_Y}{A}, \qquad (c)$$
$$\bar{y}=\frac{\Sigma y\Delta A}{A}=\frac{\Sigma\mathfrak{M}_X}{A}; \qquad (d)$$

$\mathfrak{M}$ = Static moment about given axis.



In Fig 2 let A_1, A_2, A_3 and A_4 = areas of top Ls, bottom Ls, cov. pl. and web pls. and y_1, y_2, y_3, y_4 be ordinates of their centroids.

$\bar{x}=0$ by symmetry.
$$\bar{y}=\frac{\Sigma\mathfrak{M}_x}{A}=\frac{A_1y_1+A_2y_2+A_3y_3+A_4y_4}{A_1+A_2+A_3+A_4}$$

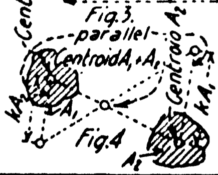
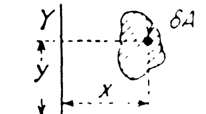


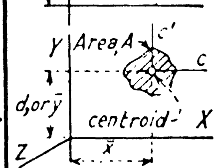
Fig 3 Centroid of trapezoid

Fig 4 Centroid of any two areas.

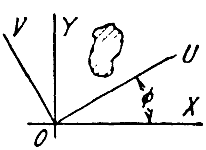
33. MOMENT OF INERTIA AND PRODUCT OF INERTIA.



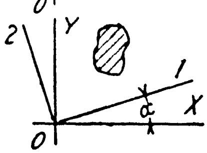
General Formulas,
$$I_X=\int y^2\delta A$$
$$I_Y=\int x^2\delta A$$
$$J_{XY}=\int xy\delta A$$
$$r_X=\sqrt{\frac{I_X}{A}}, \quad r_Y=\sqrt{\frac{I_Y}{A}},$$



Transformation Formulas,
$$I_X=I_c+Ad^2;\ I_c=I_X-Ad^2; \qquad (a)$$
$$r_X^2=r_c^2+d^2;\ r_c^2=r_X^2-d^2; \qquad (b)$$
$$J_{XY}=J_{cc'}+A\bar{x}\bar{y}, \qquad (c)$$
$$I_Z=I_X+I_Y \qquad (d)$$
$$r_Z^2=r_X^2+r_Y^2 \qquad (e)$$



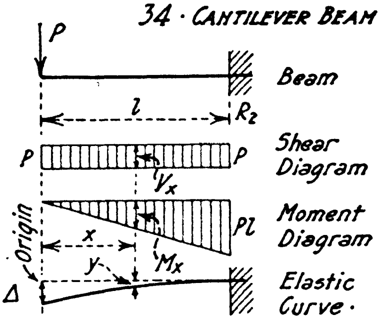
$$I_U+I_V=I_X+I_Y \qquad (f)$$
$$I_U=I_X\cos^2\phi+I_Y\sin^2\phi-J_{XY}\sin 2\phi$$
$$J_{UV}=\frac{1}{2}(I_X-I_Y)\sin 2\phi+J_{XY}\cos 2\phi$$



Principal Moments of Inertia;
$$\tan 2\alpha=2J_{XY}/(I_Y-I_X); \qquad (i)$$
$$I_1=I_X\cos^2\alpha+I_Y\sin^2\alpha+J_{XY}\sin 2\alpha$$
$$I_2=I_X+I_Y-I_1; \qquad (k)$$

Axes are designated by subscripts.

34 · CANTILEVER BEAM WITH LOAD, P, AT FREE END ·

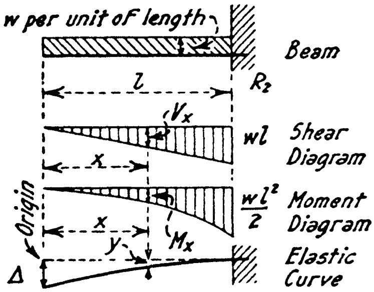


End Reaction, $R_2 = P$.
Shear at any point, $V_x = P$.
Moment at any point, $M_x = Px$.
Maximum Moment, $M = Pl$.
Equation of Elastic Curve,

$$y = \frac{P}{6EI}(2l^3 - 3l^2x + x^3)$$

$$\Delta = \frac{Pl^3}{3EI}$$

35 · CANTILEVER BEAM WITH UNIFORM LOAD, W PER UNIT OF LENGTH ·

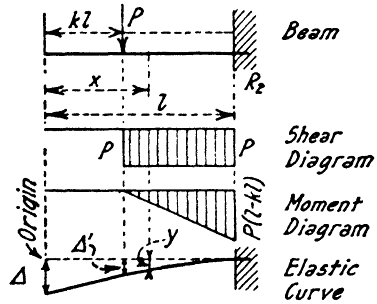


End Reaction, $R_2 = wl$.
Shear at any point $V_x = wx$.
Max. Shear, $V = wl$.
Moment at any point, $M_x = \frac{wx^2}{2}$
Max. Moment, at Right Support, $M = \frac{wl^2}{2}$
Equation of Elastic Curve

$$y = \frac{w}{24EI}(x^4 - 4l^3x + 3l^4)$$

$$\Delta = \frac{wl^4}{8EI}$$

36 · CANTILEVER BEAM WITH CONCENTRATED LOAD, P, AT ANY POINT ·



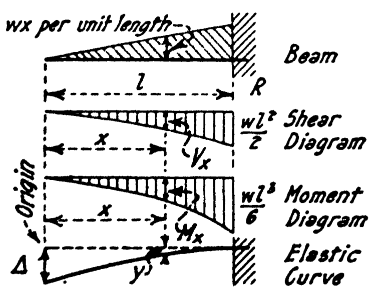
End Reaction, $R_2 = P$.
Shear between P and Support $= P$.
Moment between P and Support $= P(x - kl)$
Max. Moment, at Right Support $= P(l - kl)$
Equation of Elastic Curve between P & R_2

$$y = \frac{P}{6EI}(-3kl^3 + 2l^3 + x^3 - 3klx^2 - 3l^2x + 6kl^2x)$$

Deflection under Load, $\Delta' = \frac{P}{3EI}(l - kl)^3$

Max. Deflection, $\Delta = \frac{Pl^3}{6EI}(2 - 3k + k^3)$

37 CANTILEVER BEAM WITH VARIABLE LOAD

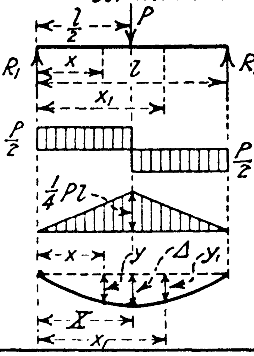


End Reaction, $R_2 = \frac{wl^2}{2}$.
Shear at any point, $V_x = \frac{wx^2}{2}$.
Max. Shear, $V = \frac{wl^2}{2}$.
Moment at any point, $M_x = \frac{wx^3}{6}$.
Max. Moment, $M = \frac{wl^3}{6}$.
Equation of Elastic Curve

$$y = \frac{w}{120EI}(x^5 - 5l^4x + 4l^5)$$

Max. Deflection, $\Delta = \frac{wl^5}{30EI}$

38. SIMPLE BEAM—CONCENTRATED LOAD AT THE CENTER.



Beam

Shear Diagram

Moment Diagram

Elastic Curve

End Reactions; $R_1 = R_2 = \frac{P}{2}$.

Shear at any point:

Between R_1 & P and between P & R_2. $V_x = \frac{P}{2}$.

Max. Shear, $V = \frac{P}{2}$.

Moment at any point:

Between R_1 & P; $M_x = R_1 x = \frac{P}{2} x$.

Between P & R_2; $M_x = R_1 x_1 - P\left(x_1 - \frac{l}{2}\right) = \frac{P}{2}(l - x_1)$

Max. Moment; $M = \frac{1}{4} Pl$, occurs at $x = \frac{l}{2}$.

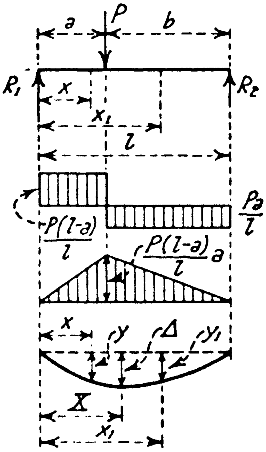
Elastic Curve and Deflections:

Between R_1 & P; $y = \frac{P}{48EI}(4x^3 - 3l^2 x)$.

Between P & R_2; symmetrical about center.

Max. Deflection; $\Delta = \frac{1}{48}\frac{Pl^3}{EI}$, $X = \frac{l}{2}$.

39. SIMPLE BEAM—CONCENTRATED LOAD AT ANY POINT.



Beam

Shear Diagram

Moment Diagram

Elastic Curve

End Reactions: $R_1 = \frac{P(l-a)}{l}$; $R_2 = \frac{Pa}{l}$.

Shear at any point:

Between R_1 & P, $V_x = R_1 = \frac{P(l-a)}{l}$.

Between P & R_2, $V_x = R_2 = \frac{Pa}{l}$.

Max. Shear; for $a < \frac{l}{2}$, $V = R_1$; for $a > \frac{l}{2}$, $V = R_2$.

Moment at any point:

Between R_1 & P; $M_x = R_1 x = \frac{P(l-a)}{l} x$.

Between P & R_2; $M_x = R_1 x - P(x_1 - a) = \frac{P(l-a)}{l} x - P(x-a)$

Max. Moment; $M = R_1 a = \frac{P(l-a)}{l} a$; occurs at $x = a$.

Elastic Curve and Deflections:

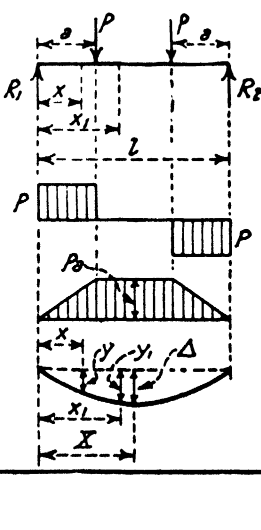
Between R_1 & P; $y = \frac{Px(l-a)}{6EIl}(2la - a^2 - x^2)$.

Between P & R_2; $y_1 = \frac{Pa(l-x_1)}{6EIl}(2lx_1 - a^2 - x_1^2)$.

Max. Defl.; $a < \frac{l}{2}$; $\Delta = \frac{P(l-b)}{3EIl}\left[\frac{b(2l-b)}{3}\right]^{\frac{3}{2}}$; $X = l - \sqrt{\frac{b(2l-b)}{3}}$

Max. Defl.; $a > \frac{l}{2}$; $\Delta = \frac{P(l-a)}{3EIl}\left[\frac{a(2l-a)}{3}\right]^{\frac{3}{2}}$; $X = \sqrt{\frac{a(2l-a)}{3}}$

40. SIMPLE BEAM—TWO EQUAL CONCENTRATED LOADS, SYMMETRICALLY PLACED.



Beam

Shear Diagram

Moment Diagram

Elastic Curve

End Reactions; $R_1 = R_2 = P$.

Shear at any point:

Between R_1 and left P; $V_x = P$.

Between Loads; $V_x = 0$.

Between right P and R_2; $V_x = P$.

Max. Shear, $V = P$.

Moment at any point:

Between R_1 and left P; $M_x = Px$.

Between Loads; $M_x = R_1 x - P(x-a) = Pa$.

Max. Moment; $M = Pa$.

Elastic Curve & Deflections:

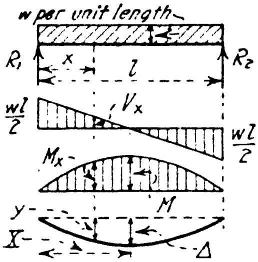
Between R_1 & left P; $y = \frac{Px}{6EI}(3la - 3a^2 - x^2)$.

Between Loads; $y_1 = \frac{Pa}{6EI}(3lx - 3x^2 - a^2)$.

Between right P & R_2; symmetrical with left load & R_1.

Max. Deflection; $\Delta = \frac{Pa}{24EI}(3l^2 - 4a^2)$; $X = \frac{l}{2}$.

41. SIMPLE BEAM – UNIFORM LOAD.



Beam
Shear Diagram
Moment Diagram
Elastic Curve

End Reactions: $R_1 = R_2 = \frac{wl}{2}$.

Shear at any point: $V_x = \frac{wl}{2} - wx$.

Max Shear; $V = \frac{wl}{2}$; occurs at each support

Moment at any point: $M_x = \frac{wl}{2}x - \frac{1}{2}wx^2$

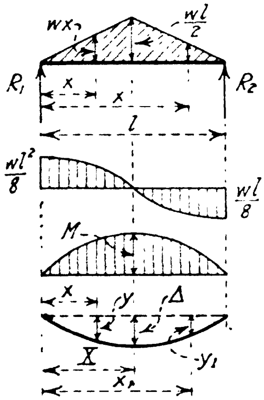
Max Moment; $M = \frac{1}{8}wl^2$, occurs at center.

Elastic Curve and Deflections:

$$y = \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$$

Max Deflection; $\Delta = \frac{5}{384}\frac{wl^4}{EI}$; $X = \frac{l}{2}$.

42. SIMPLE BEAM – TRIANGULAR LOAD WITH MAXIMUM AT THE CENTER



Beam
Shear Diagram
Moment Diagram
Elastic Curve

Total Load $= \frac{wl^2}{4}$.

End Reactions: $R_1 = R_2 = \frac{wl^2}{8}$

Shear at any point:

Between R_1 & Center; $V_x = w\left(\frac{l^2}{8} - \frac{x^2}{2}\right)$.

Between Center & R_2; $V_x = w\left(\frac{3}{8}l^2 - lx + \frac{x^2}{2}\right)$

Max Shear; $V = \frac{1}{8}wl^2$, occurs at supports.

Moment at any point:

Between R_1 and Center; $M_x = wx\left(\frac{l^2}{8} - \frac{x^2}{6}\right)$

Between Center & R_2; $M_x = \frac{w}{24}(-l^3 + 9l^2x - 12lx^2 + 4x^3)$

Max Moment; $M = \frac{1}{24}wl^3$; occurs at center.

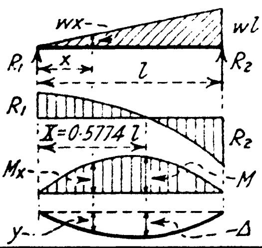
Elastic Curve and Deflections:

Between R_1 & Center: $y = \frac{wx}{24EI}\left[\frac{l^2x^2}{2} - \frac{x^4}{5} - \frac{5l^4}{16}\right]$

Between Center & R_2; Symmetrical.

Max Deflection; $\Delta = \frac{wl^5}{240EI}$; $X = \frac{l}{2}$.

43. SIMPLE BEAM – TRIANGULAR LOAD WITH MAXIMUM AT RIGHT END.



Beam
Shear Diagram
Moment Diagram
Elastic Curve

Total Load $= \frac{wl^2}{2}$.

End Reactions: $R_1 = \frac{1}{6}wl^2$; $R_2 = \frac{1}{3}wl^2$.

Shear at any point: $V_x = \frac{w}{2}\left(\frac{l^2}{3} - x^2\right)$

Max Shear; $V = \frac{1}{3}wl^2$ occurs at right support.

Moment at any point: $M_x = \frac{wx}{6}(l^2 - x^2)$.

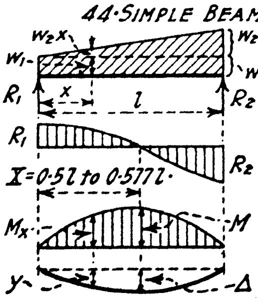
Max Moment; $M = 0.064\,wl^3$ occurs at $X = 0.5774\,l$.

Elastic Curve and Deflections:

$$y = \frac{wx}{12EI}\left(-\frac{l^2x^2}{3} + \frac{x^4}{10} + \frac{7}{30}l^4\right)$$

Max Deflection; $\Delta = 0.00652\frac{wl^5}{EI}$, $X = 0.51913\,l$.

44. SIMPLE BEAM – TRAPEZOIDAL LOAD WITH MAXIMUM AT RIGHT END.



Beam
Shear Diagram
Moment Diagram
Elastic Curve.

Total Load $= w_1l + \frac{w_2l^2}{2}$.

End Reactions $= R_1 = \frac{l}{2}\left(w_1 + \frac{w_2l}{3}\right)$; $R_2 = \frac{l}{2}\left(w_1 + \frac{2}{3}w_2l\right)$

Shear at any point: $V_x = w_1\left(\frac{l}{2} - x\right) + \frac{w_2}{2}\left(\frac{l^2}{3} - x^2\right)$

Max Shear; $V = \frac{l}{2}\left(w_1 + \frac{2}{3}w_2l\right)$, occurs at right support.

Moment at any point; $M_x = \frac{w_1}{2}(lx - x^2) + \frac{w_2}{6}(l^2x - x^3)$

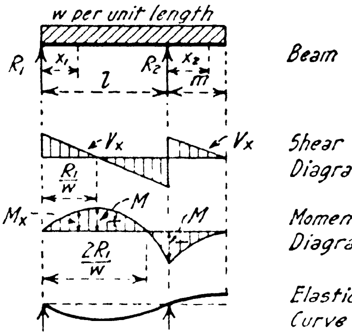
Max Moment; $M = \left(w_1l + \frac{w_2}{2}l^2\right)\frac{l}{8}$ (Approx.)

Elastic Curve and Deflections:

$$y = \frac{w_1x}{24EI}(l^3 - 2lx^2 + x^3) - \frac{w_2x}{12EI}\left(\frac{l^2x^2}{3} - \frac{x^4}{10} - \frac{7}{30}l^4\right)$$

Max Defl.; $\Delta = \frac{5}{384}\frac{wl^4}{EI} + 0.00652\frac{w_2l^5}{EI}$; $X = 0.5l$ (Approx.)

45. BEAM OVER-HANGING ONE SUPPORT – UNIFORM LOAD.



Reactions: $R_1 = \frac{1}{2}wl - \frac{1}{2}wm\left(\frac{m}{l}\right)$; $R_2 = \frac{1}{2}wl + wm + \frac{1}{2}wm\left(\frac{m}{l}\right)$

Shear at any point:

Between R_1 & R_2; $V_x = R_1 - wx_1$.

Between R_2 and End; $V_x = w(m - x_2)$

Moment at any point:

Between R_1 & R_2; $M_x = R_1x_1 - \frac{1}{2}wx^2$.

Between R_2 & End; $M_x = \frac{w}{2}(m - x_1)^2$.

Max. Positive Moment; $M = \frac{R_1^2}{2w}$; occurs when $x = \frac{R_1}{w}$.

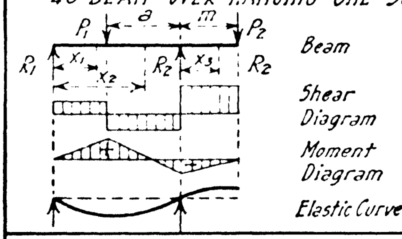
Max. Negative Moment; $M = \frac{1}{2}wm^2$; occurs at $x = R_2$.

Elastic Curve and Deflections.

Between R_1 & R_2; $y = \frac{1}{24EI}\left[4R_1(x_1^3 - l^2x_1) - w(x_1^4 - l^3x_1)\right]$

Between R_2 & End; $y = \frac{1}{24EI}\left[w(6m^2x_2^2 - 4mx_2^3 + 3l^3x_2 + x_2^4) - 8R_1l^2x_2\right]$

46. BEAM OVER-HANGING ONE SUPPORT – CONCENTRATED LOAD AT ANY POINT.



Reactions; $R_1 = \frac{P_1a - P_2m}{l}$; $R_2 = \frac{P_1(l - a) + P_2(m + l)}{l}$.

Shear at any point:

Between R_1 & P_1: $V_x = R_1$; between P_1 & R_2; $V_x = R_1 - P_1$;

Between R_2 & P_2; $V_x = P_2$.

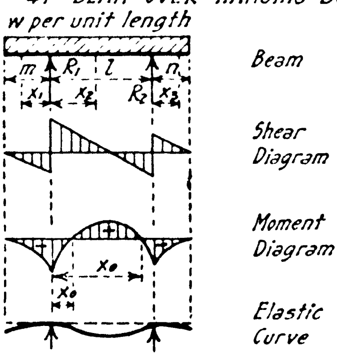
Moment at any point:

Between R_1 & P_1; $M_x = R_1x_1$.

Between P_1 & R_2; $M_x = R_1x_2 - P_1(a + x_2 - l)$

Between R_2 & P_2; $M_x = P_2(m - x_3)$.

47. BEAM OVER-HANGING BOTH SUPPORTS – UNIFORM LOAD.



Reactions: $R_1 = \frac{w}{2l}\left[(m + l)^2 - n^2\right]$; $R_2 = \frac{w}{2l}\left[(n + l)^2 - m^2\right]$

Shear at any point:

Between left end & R_1; $V_x = w(m - x_1)$

Between R_1 & R_2; $V_x = R_1 - w(m + x_2)$

Between R_2 & right end; $V_x = w(n - x_3)$

Max. Shear; $V = wm$, or $R_1 - wm$.

Moment at any point:

Between left end & R_1; $M_x = \frac{1}{2}w(m - x_1)^2$.

Between R_1 & R_2; $M_x = \frac{1}{2}w(m + x_2)^2 - R_1x_2$.

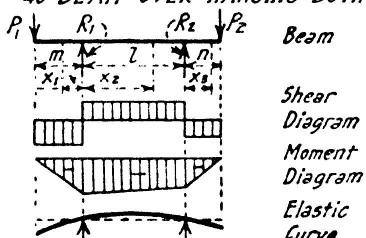
Between R_2 & right end; $M_x = \frac{1}{2}w(n - x_3)^2$.

Max. Positive Moment; $M = R_1\left(\frac{R_1}{2w} - m\right)$, occurs at $x_2 = \frac{R_1}{w} - m$.

Max. Negative Moments; $M' = \frac{1}{2}wm^2$ at R_1; $M' = \frac{1}{2}wn^2$ at R_2

Points of Contraflexure; $x_0 = \left(\frac{R_1}{w} - m\right) \pm \sqrt{\left(\frac{R_1}{w}\right)^2 - \frac{2R_1}{w}m}$.

48. BEAM OVER-HANGING BOTH SUPPORTS – TWO EXTERIOR CONCENTRATED LOADS.



Reactions. $R_1 = \frac{P_1m - P_2n}{l} + P_1$; $R_2 = \frac{P_2n - P_1m}{l} + P_2$.

Shear at any point:

Between P_1 & R_1; $V_x = P_1$: R_1 & R_2, $V_x = P_1 - R_1$; R_2 & P_2, $V_x = P_2$.

Moment at any point:

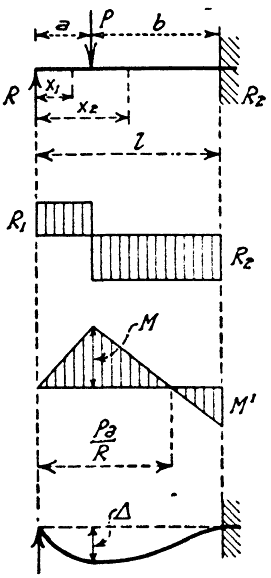
Between P_1 & R_1; $M_x = P_1(m - x_1)$

Between R_1 & R_2; $M_x = P_1m + (P_1 - R_1)x_2$.

Between R_2 & P_2; $M_x = P_2(n - x_3)$

Moment at R_1; $M = P_1m$; at R_2, $M = P_2n$.

49. BEAM FIXED AT ONE END AND SUPPORTED AT OTHER—CONCENTRATED LOAD AT ANY POINT.



End Reactions: $R = P\left[\frac{3b^2l - b^3}{2l^3}\right]$; $R_2 = P\left[\frac{3al^2 - a^3}{2l^3}\right]$

Shear at any point: Between R_1 & P, $V_x = R_1$; between P & R_2, $V_x = R_1 - P$.

Moment at any point: Between R & P: $M_x = R_1x$; between P & R_2: $M_x = R_1x_2 - P(x_2 - a)$

Max. Positive Moment: $M = R_1a$, occurs under load.

Max. Negative Moment: $M' = R_1l - P(l - a)$, occurs at fixed end.

Point of Contraflexure: $x_0 = \frac{Pa}{R_2}$.

Elastic Curve & Deflections:

Between R_1 & P; $y = \frac{1}{6EI}[3R_1l^2x - R_1x^3 - 3P(l-a)^2x]$

Between P & R_2; $y = \frac{1}{6EI}[R_2(2l^3 - 3l^2x + x^3) - 3Pa(l-x)^2]$

For $a = 0.414\,l$; Max. Defl. $\Delta = 0.0098\frac{Pl^3}{EI}$, occurs under load.

IF P IS A MOVING LOAD:

Absolute End Reactions:

$R_1 = P$, occurs when $a = 0$; $R_2 = P$, occurs when $a = l$.

Absolute Maximum Shears:

$R_1 = P$; occurs when $a = 0$, at $x = 0$; $R_2 = P$, occurs when $a = l$, at $x = l$.

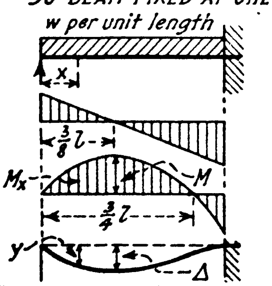
Absolute Maximum Moments:

Max. Moment is Negative and is $M = 0.1925\,Pl$; occurs at fixed end when $a = 0.5774\,l$.

Absolute Maximum Deflection:

$\Delta = 0.0098\frac{Pl^3}{EI}$, occurs under load when $a = 0.414\,l$.

50. BEAM FIXED AT ONE END AND SUPPORTED AT OTHER — UNIFORM LOAD.



End Reactions: $R_1 = \frac{3}{8}wl$; $R_2 = \frac{5}{8}wl$.

Shear at any point: $V_x = w(\frac{3}{8}l - x)$.

Max. Shear; $V = \frac{5}{8}wl$, occurs at right support.

Moment at any point: $M_x = wx(\frac{3}{8}l - \frac{1}{2}x)$

Max. Positive Moment; $M = \frac{9}{128}wl^2$, occurs at $x = \frac{3}{8}l$.

Max. Negative Moment; $M' = \frac{1}{8}wl^2$; occurs at right support.

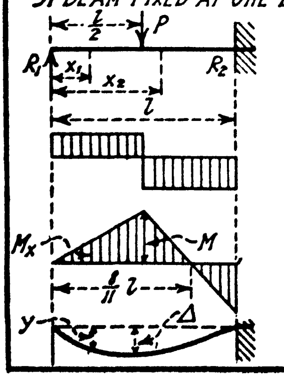
Point of Contraflexure; $x_0 = \frac{3}{4}l$.

Elastic Curve and Deflections:

$y = \frac{wx}{48EI}[-3lx^2 + 2x^3 + l^3]$

Max. Deflection; $\Delta = 0.0054\frac{wl^4}{EI}$, $X = 0.4215\,l$.

51. BEAM FIXED AT ONE END SUPPORTED AT OTHER—CONCENTRATED LOAD AT CENTER.



End Reactions: $R_1 = \frac{5}{16}P$; $R_2 = \frac{11}{16}P$.

Shear at any point:

Between R_1 & P; $V_x = \frac{5}{16}P$; Between P & R_2; $V_x = \frac{11}{16}P$.

Max. Shear; $V = \frac{11}{16}P$, occurs at R_2.

Moment at any point:

Between R_1 & P; $M_x = \frac{5}{16}P$; Between P & R_2; $M_x = \frac{1}{2}Pl - \frac{11}{16}Px$.

Max. Positive Moment: $M = \frac{5}{32}Pl$, occurs under load.

Max. Negative Moment: $M' = \frac{3}{16}Pl$, occurs at fixed end.

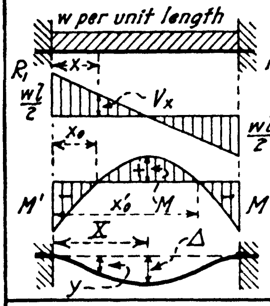
Elastic Curve & Deflections:

Between R_1 & P; $y = \frac{Px_1}{96EI}(5x_1^2 - 3l^2)$.

Between P & R_2; $y = \frac{P}{96EI}(-2l^3 + 15l^2x_2 - 24lx_2^2 + 11x_2^3)$

Max Deflection; $\Delta = 0.00932\frac{Pl^3}{EI}$; $X = 0.4472\,l$.

52. BEAM FIXED AT BOTH ENDS — UNIFORM LOAD.



Beam

Shear Diagram

Moment Diagram

Elastic Curve

End Reactions: $R_1 = R_2 = \frac{1}{2}wl$.

Shear at any point: $V_x = \frac{1}{2}wl - wx$.

Max. Shear; $V = \frac{1}{2}wl$, occurs at supports.

Moment at any point: $M_x = \frac{w}{2}\left(-\frac{1}{6}l^2 + lx - x^2\right)$.

Max. Positive Moment; $M = \frac{1}{24}wl^2$, occurs at center.

Max. Negative Moment; $M' = \frac{1}{12}wl^2$, occurs at supports.

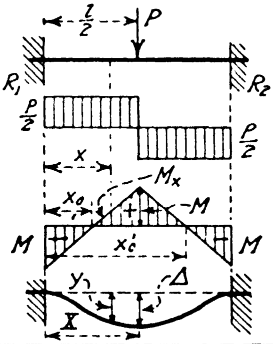
Points of Contraflexure; $x_0 = 0{\cdot}2113\,l$; $x'_0 = 0{\cdot}7887\,l$.

Elastic Curve and Deflections:

$y = \frac{wx^2}{24EI}(l^2 - 2lx + x^2)$.

Max. Defl.; $\Delta = \frac{1}{384}\frac{wl^4}{EI}$, $X = \frac{l}{2}$.

53. BEAM FIXED AT BOTH ENDS — CONCENTRATED LOAD AT CENTER.



Beam

Shear Diagram

Moment Diagram

Elastic Curve

End Reactions: $R_1 = R_2 = \frac{1}{2}P$.

Shear at any point: $V_x = \frac{1}{2}P$. Max. Shear, $V = \frac{1}{2}P$.

Moment at any point:

Between R_1 & P; $M_x = \frac{1}{2}P(x - \frac{1}{4}l)$.

Between P & R_2; $M_x = \frac{1}{2}P(\frac{3}{4}l - x)$.

Max. Positive Moment; $M = \frac{1}{8}Pl$, occurs at center.

Max. Negative Moment; $M = \frac{1}{8}Pl$, occurs at supports.

Points of Contraflexure; $x_0 = \frac{l}{4}$; $x'_0 = \frac{3}{4}l$.

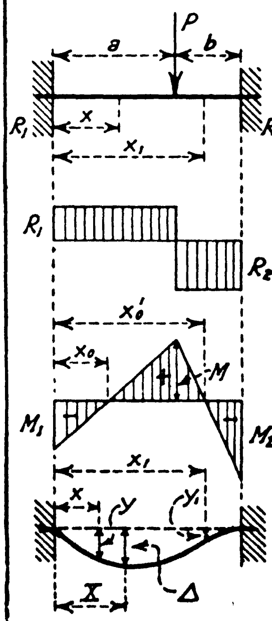
Elastic Curve and Deflections:

Between R_1 & P; $y = \frac{Px^2}{6EI}\left(-\frac{1}{2}x + \frac{3}{8}l\right)$.

Between P & R_2; Symmetrical.

Max. Defl.; $\Delta = \frac{1}{192}\frac{Pl^3}{EI}$; $X = \frac{l}{2}$.

54. BEAM FIXED AT BOTH ENDS — CONCENTRATED LOAD AT ANY POINT.



Beam

Shear Diagram

Moment Diagram

Elastic Curve

End Reactions: $R_1 = P\frac{b^2(3a+b)}{l^3}$; $R_2 = P\frac{a^2(3b+a)}{l^3}$

Shear at any point: Between R_1 & P; $V_x = R_1$; Between P & R_2; $V_x = R_2$

Max. Shear; $V = R_1$ for $a < b$; $V = R_2$ for $a > b$.

Moment at any point:

Negative Moments at Supports; $M_1 = -P\frac{ab^2}{l^2}$; $M_2 = -P\frac{a^2b}{l^2}$.

Between R_1 & P; $M_x = R_1x + M_1$. } Note that M_1 carries

Between P & R_2; $M_x = R_1x_1 + M_1 - P(x_1 - a)$ } a minus sign.

Max Positive Moment: $M = R_1a + M_1$; occurs under load.

Max. Negative Moments occur at supports: See above.

Points of Contraflexure; $x_0 = \frac{al}{3a+b}$; $x'_0 = l - \frac{bl}{3b+a}$.

Elastic Curve and Deflections:

Between R_1 & P; $y = \frac{Pb^2x^2}{6EIl^3}\left[3al - 3ax - bx\right]$.

Between P and R_2; $y_1 = \frac{Pb^2x_1^2}{6EIl^3}\left[\frac{l^3(x_1-a)^3}{b^3x_1^2} + 3al - 3ax_1 - bx_1\right]$.

Max. Defl., when $a > b$; $\Delta = \frac{2Pa^3b^2}{3EI(3a+b)^2}$ occurs at $X = \frac{2al}{3a+b}$.

Max. Defl.; when $a < b$; $\Delta = -\frac{2Pb^3a^2}{3EI(3b+a)^2}$; occurs at $X = \frac{l^2}{3b+a}$

IF P IS A MOVING LOAD:

Absolute Max. Shears; $S = P$, occurs at R_1 when $a = 0$; at R_2 when $a = l$.

Absolute Max. Negative Moment; $M_1 = \frac{4}{27}Pl$; occurs when $a = \frac{1}{3}l$.

Absolute Max. Negative Moment; $M_2 = \frac{4}{27}Pl$; occurs when $a = \frac{2}{3}l$.

Absolute Max. Positive Moment; $M = \frac{1}{8}Pl$; occurs when $a = \frac{1}{2}l$.

Absolute Max. Deflection; $\Delta = \frac{1}{192}\frac{Pl^3}{EI}$; occurs when $a = \frac{l}{2}$.

55. MAXIMUM SHEARS AND MOMENTS IN SIMPLE BEAMS FOR MOVING CONCENTRATED LOADS.

Criterion for Maximum Shear.

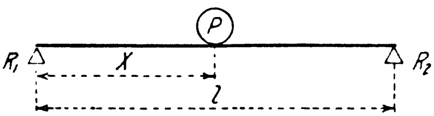
The maximum shear due to moving concentrated loads will occur at one support when one of the loads is at that support and will equal the total reaction. The load giving the maximum must be determined by trial.

Criterion for Maximum Moment.

The maximum moment due to moving concentrated loads will occur under one of the loads when that load is as far from one end as the center of gravity of all the loads on the beam is from the other end. The load giving the greatest maximum must be found by trial.

For beams fixed at one or both ends and carrying one load, see 49 and 54, in this chapter.

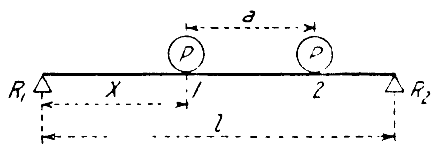
a. ONE LOAD.



Max. Shear, $X=0$; $V=P$; at R_1.

Max. Moment, $X=\frac{1}{2}l$ $M=\frac{1}{4}Pl$; at P.

b. TWO EQUAL LOADS.

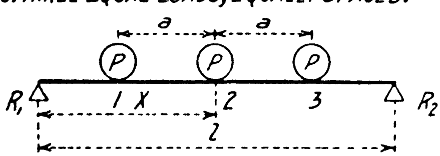


Max. Shear, $X=0$; $V=P+P\frac{l-a}{l}$; at R_1

Max. Moment, $X=\frac{1}{2}(l-\frac{a}{2})$; $M=P\frac{(l-\frac{a}{2})^2}{2l}$; at 1.

If a is greater than $0.586\,l$, one load gives max. M as in a.

c. THREE EQUAL LOADS, EQUALLY SPACED.

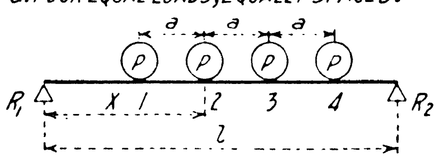


Max. Shear, $X=a$; $V=3P\frac{l-a}{l}$; at R_1.

Max. Moment, $X=\frac{1}{2}l$; $M=P(\frac{3}{4}l-a)$; at 2.

If a is greater than $0.450\,l$, two loads give max. M as in b.

d. FOUR EQUAL LOADS, EQUALLY SPACED.

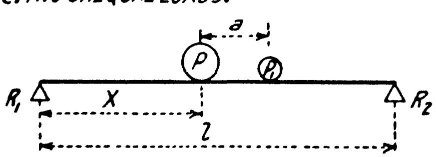


Max. Shear, $X=a$; $V=4P\frac{l-\frac{3}{2}a}{l}$; at R.

Max. Moment, $X=\frac{1}{2}(l-\frac{1}{2}a)$; $M=P(l-2a+\frac{a^2}{4l})$; at 2.

If a is greater than $0.268\,l$, three loads give max. M as in c.

e. TWO UNEQUAL LOADS.

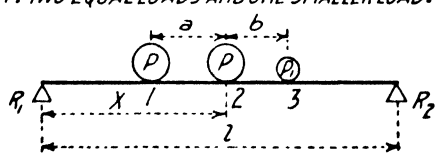


Max. Shear, $X=0$; $V=P+P_1\cdot\frac{l-a}{l}$; at R_1.

Max. Moment, $X=\frac{1}{2}\left[l-\frac{P_1a}{P+P_1}\right]$; $M=[P+P_1]\frac{X^2}{l}$; at P.

Max. moment may occur for one load as in a.

f. TWO EQUAL LOADS AND ONE SMALLER LOAD.

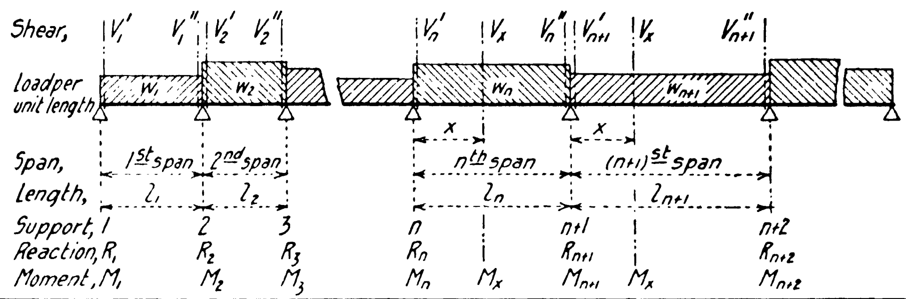


Max. Shear, $X=a$; $V=P+\left[\frac{P(l-a)+P_1(l-a-b)}{l}\right]$; at R_1

Max. Moment, $X=\frac{1}{2}\left[l-\frac{P_1b-Pa}{P_1+2P}\right]$; $M=\frac{P_1+2P}{l}X^2-Pa$; at 2.

Max. moment may occur for two equal loads as in b.

56. CONTINUOUS BEAMS, UNIFORM LOADS, CONSTANT MOMENT OF INERTIA AND MODULUS OF ELASTICITY.



Relation between moments at supports for the n^{th} and $(n+1)^{st}$ spans,

$$M_n l_n + 2M_{n+1}(l_n + l_{n+1}) + M_{n+2} l_{n+1} = -\frac{1}{4} w_n l_n^3 - \frac{1}{4} w_{n+1} l_{n+1}^3; \quad (a)$$

Shear to right of n^{th} support,

$$V_n' = \frac{M_{n+1} - M_n}{l_n} + \frac{1}{2} w_n l_n; \quad (b)$$

Shear to left of $(n+1)^{st}$ support,

$$V_n'' = \frac{M_{n+1} - M_n}{l_n} - \frac{1}{2} w_n l_n; \quad (c)$$

Shear to right of $(n+1)^{st}$ support,

$$V_{n+1}' = \frac{M_{n+2} - M_{n+1}}{l_{n+1}} + \frac{1}{2} w_{n+1} l_{n+1}; \quad (d)$$

Reaction at $(n+1)^{st}$ support,

$$R_{n+1} = V_{n+1}' - V_n''; \quad (\text{Note } R_1 = V_1') \quad (e)$$

Shear at any point in n^{th} span,

$$V_x = V_n' - w_n x; \quad (f)$$

Moment at any point in n^{th} span,

$$M_x = M_n + V_n' x - \frac{1}{2} w_n x^2; \quad (g)$$

Point of max. positive moment in n^{th} span,

$$x = \frac{V_n'}{w_n}; \quad (h)$$

Maximum positive moment in n^{th} span,

$$M = M_n + \frac{V_n'^2}{2w_n}; \quad (i)$$

EXPLANATION OF FORMULAS; n = number of first span considered or its left support.

Given a continuous beam of several spans uniformly loaded (for spans with no load $w=0$), Apply formula (a) to 1^{st} and 2^{nd} spans at the left end making $n=1$. Three unknown moments appear, M_1, M_2, and M_3. If beam is simply supported at left end $M_1 = 0$. Next apply formula (a) to 2^{nd} and 3^{rd} spans making $n=2$. Again there will be three unknowns M_2, M_3 and M_4. Continue until last two spans have been considered (never consider last span alone). If beam is simply supported at right end, the M for that support $=0$. There are now as many equations as there are unknowns so by solving, the moments at all of the supports may be found. If the beam is symmetrical as to loading and dimensions, the calculations may be shortened by equating moments which are known, by inspection, to be equal. Knowing the moments at the supports; the shear at any point, the reactions, and the moment at any point may be calculated. ($R_1 = V_1'$ and R for last support equals V'' for last span). For fixed ends imagine the beam to extend one span beyond the fixed end and apply the formulas, as above, equating the length and load of the imaginary span to zero and the moment at the extreme end of the imaginary span to zero. Care should be taken that shears and moments are used with their proper sign.

SPECIAL CASES;

For a beam of equal spans with equal uniform loads, Formula (a) reduces to-

$$M_n + 4M_{n+1} + M_{n+2} = -\frac{1}{2} w l^2; \quad \text{(See also 57, of this chapter.)} \quad (j)$$

For a beam of two unequal spans with unequal uniform loads and simply supported at the ends, $M_1 = 0$, $M_3 = 0$ and from formula (a)

$$M_2 = -\frac{\frac{1}{4} w_1 l_1^3 + \frac{1}{4} w_2 l_2^3}{2(l_1 + l_2)} \quad (k)$$

57. MOMENTS AT SUPPORTS: CONTINUOUS BEAMS, EQUAL SPANS AND EQUAL UNIFORM LOADS.

Number of Spans									Number of Spans
1.	0	0							1.
2.	0	$-\frac{1}{8}$	0						2.
3.	0	$-\frac{1}{10}$	$-\frac{1}{10}$	0					3.
4.	0	$-\frac{3}{28}$	$-\frac{2}{28}$	$-\frac{3}{28}$	0				4.
5.	0	$-\frac{4}{38}$	$-\frac{3}{38}$	$-\frac{3}{38}$	$-\frac{4}{38}$	0			5.
6.	0	$-\frac{11}{104}$	$-\frac{8}{104}$	$-\frac{9}{104}$	$-\frac{8}{104}$	$-\frac{11}{104}$	0		6.
7.	0	$-\frac{15}{142}$	$-\frac{11}{142}$	$-\frac{12}{142}$	$-\frac{12}{142}$	$-\frac{11}{142}$	$-\frac{15}{142}$	0,	7.

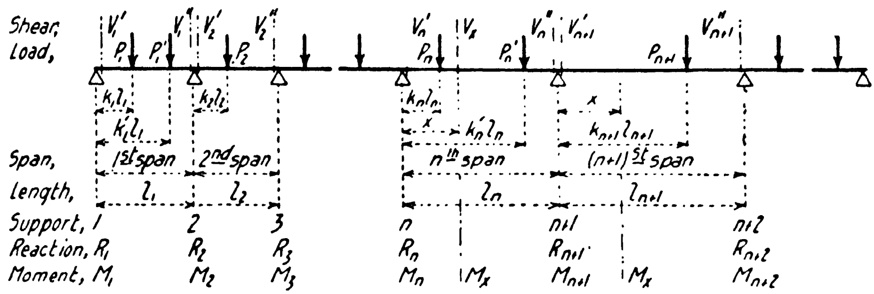
COEFFICIENTS OF wl^2, where w = load per unit length and l = length of one span. E and I constant.
Maximum positive moment in any span can be calculated from formula 56 *i*.

58. SHEARS AT SUPPORTS: CONTINUOUS BEAMS, EQUAL SPANS AND EQUAL UNIFORM LOADS.

Number of Spans									Number of Spans
1.	$0, +\frac{1}{2}$	$-\frac{1}{2}, 0$							1.
2.	$0, +\frac{3}{8}$	$-\frac{5}{8}, +\frac{5}{8}$	$-\frac{3}{8}, 0$						2.
3.	$0, +\frac{4}{10}$	$-\frac{6}{10}, +\frac{5}{10}$	$-\frac{5}{10}, +\frac{6}{10}$	$-\frac{4}{10}, 0$					3.
4.	$0, +\frac{11}{28}$	$-\frac{17}{28}, +\frac{15}{28}$	$-\frac{13}{28}, +\frac{13}{28}$	$-\frac{15}{28}, +\frac{17}{28}$	$-\frac{11}{28}, 0$				4.
5.	$0, +\frac{15}{38}$	$-\frac{23}{38}, +\frac{20}{38}$	$-\frac{18}{38}, +\frac{19}{38}$	$-\frac{19}{38}, +\frac{18}{38}$	$-\frac{20}{38}, +\frac{23}{38}$	$-\frac{15}{38}, 0$			5.
6.	$0, +\frac{41}{104}$	$-\frac{63}{104}, +\frac{55}{104}$	$-\frac{49}{104}, +\frac{51}{104}$	$-\frac{53}{104}, +\frac{53}{104}$	$-\frac{51}{104}, +\frac{49}{104}$	$-\frac{55}{104}, +\frac{63}{104}$	$-\frac{41}{104}, 0$		6.
7.	$0, +\frac{56}{142}$	$-\frac{86}{142}, +\frac{75}{142}$	$-\frac{67}{142}, +\frac{70}{142}$	$-\frac{72}{142}, +\frac{71}{142}$	$-\frac{71}{142}, +\frac{72}{142}$	$-\frac{70}{142}, +\frac{67}{142}$	$-\frac{75}{142}, +\frac{86}{142}$	$-\frac{56}{142}, 0$,	7.

COEFFICIENTS OF wl, where w = load per unit length and l = length of span. E and I constant.
Reactions at supports equal algebraic difference of shears to right and left.

59. CONTINUOUS BEAMS, CONCENTRATED LOADS, CONSTANT MOMENT OF INERTIA AND MODULUS OF ELASTICITY.



Relation between moments at supports for n^{th} and $(n+1)^{st}$ spans,

$$M_n l_n + 2M_{n+1}(l_n + l_{n+1}) + M_{n+2} l_{n+1} = -\Sigma[P_n l_n^2 (k_n - k_n^3)] - \Sigma[P_{n+1} l_{n+1}^2 (2k_{n+1} - 3k_{n+1}^2 + k_{n+1}^3)], \quad (a)$$

Shear to the right of n^{th} support,

$$V_n' = \frac{M_{n+1} - M_n}{l_n} + \Sigma[P_n(1 - k_n)]; \quad (b)$$

Shear to left of $(n+1)^{st}$ support,

$$V_n'' = \frac{M_{n+1} - M_n}{l_n} - \Sigma[P_n k_n] \quad (c)$$

Shear to right of $(n+1)^{st}$ support,

$$V_{n+1}' = \frac{M_{n+2} - M_{n+1}}{l_{n+1}} + \Sigma[P_{n+1}(1 - k_{n+1})] \quad (d)$$

Reaction at $(n+1)^{st}$ support,

$$R_{n+1} = V_{n+1}' - V_n''; \text{ (Note } R_1 = V_1') \quad (e)$$

Shear at any point in n^{th} span,
$V_x = V_n' - \Sigma P_n$, where ΣP_n equals the sum of the loads between n^{th} support and point considered. (f)

Moment at any point in n^{th} span,
$M_x = M_n + V_n' x - \Sigma[P_n(x - k_n l_n)]$, where (g) $\Sigma[P_n(x - k_n l_n)]$ equals the sum of the moments of the loads, between the n^{th} support and the point considered, about the point

Point of max. positive moment in n^{th} span,
The max. positive moment occurs where shear, as calculated from (f) passes through zero. This point is always at one of the loads. (h)

Maximum positive moment in the n^{th} span,
After the point of max positive moment has be located as described in (h) the value of x thus determined is substituted in (g) and M_x determined.

EXPLANATION OF FORMULAS: (See under 56.)

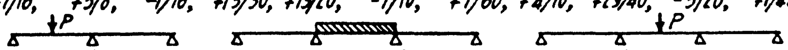
SPECIAL CASE,

For a beam of two unequal spans with unequal concentrated loads and with ends simply supported, $M_1 = 0$, $M_3 = 0$ and formula (a) reduces to-

$$M_2 = -\frac{\Sigma[P_1 l_1^2 (k_1 - k_1^3)] + \Sigma[P_2 l_2^2 (2k_2 - 3k_2^2 + k_2^3)]}{2(l_1 + l_2)}, \quad (j)$$

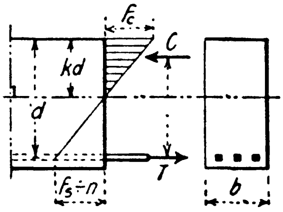
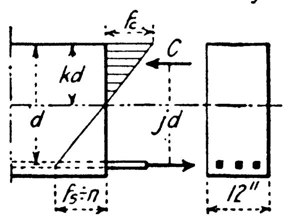
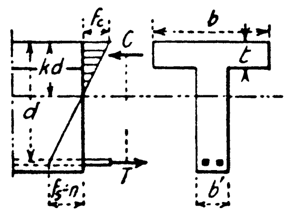
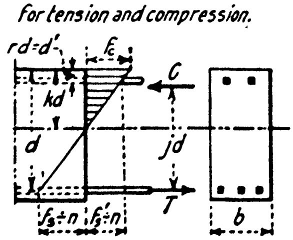
60. CONTINUOUS BEAMS OF TWO AND THREE EQUAL SPANS: Uniform load, w, per unit length or load P, in center of one span

Moment,	0,	-1/16,	0,	0,	-1/15,	+1/60,	0,	0,	-1/10,	+1/40,	0,
Reaction,	+7/16,	+5/8,	-1/16,	+13/30,	+13/20,	-1/10,	+1/60,	+4/10,	+29/40,	-3/20,	+1/40,



Moment,	0,	-3/32,	0,	0,	-1/20,	-1/20,	0,	0,	-3/40,	-3/40,	0,
Reaction,	+13/32,	+11/16,	-3/32,	-1/20,	+11/20,	+11/20,	-1/20,	-3/40,	+23/40,	+23/40,	-3/40,

Coefficients of wl^2 and Pl, for moments at supports, and of wl and P, for reactions at supports.
By addition of proper cases any beam may be solved. For shears and moments between supports see 56 & 59.

DIAGRAMS.	GENERAL FORMULAS.	FOR $f_s=16{,}000, f_c=650, n=15$.
61. RECTANGULAR BEAMS: Reinforced for tension only. 	$k=\sqrt{2pn+p^2n^2}-pn=\frac{1}{1+\frac{f_s}{nf_c}}$; $j=1-\frac{1}{3}k$; $M_s=f_sAjd=f_spjbd^2$; $M_c=\frac{1}{2}f_ckjbd^2$; $f_s=\frac{M}{Ajd}=\frac{M}{pjbd^2}$; $f_c=\frac{2M}{jkbd^2}=\frac{k}{(1-k)n}f_s=\frac{2p}{k}f_s$; Steel ratio and depth, balanced reinforcement, $p=\frac{1}{\frac{2f_s}{f_c}\left[\frac{f_s}{nf_c}+1\right]}$; $d=\sqrt{\frac{M}{f_spjb}}$;	$k=0.379$; $j=0.8737$; $M_s=107.5bd^2$; $M_c=M_s$; $f_s=16000$; $f_c=650$; Steel ratio and depth, balanced reinforcement, $p=0.0077$; $d=\sqrt{\frac{M}{107.5b}}$;
62. SLABS: Values for 12" strip. Reinforced for tension only. 	$k=\sqrt{2pn+p^2n^2}-pn=\frac{1}{1+\frac{f_s}{nf_c}}$; $j=1-\frac{1}{3}k$; $M_s=f_sAjd=12f_spjd^2$; $M_c=6f_ckjd^2$; $f_s=\frac{M}{Ajd}=\frac{M}{12pjd^2}$; $f_c=\frac{M}{6jkd^2}=\frac{k}{(1-k)n}f_s=\frac{2p}{k}f_s$; Steel ratio and depth, balanced reinforcement, $p=\frac{1}{\frac{2f_s}{f_c}\left[\frac{f_s}{nf_c}+1\right]}$ $d=\sqrt{\frac{M}{12f_spj}}$;	$k=0.379$; $j=0.8737$; $M_s=1290d^2$; $M_c=M_s$; $f_s=16000$; $f_c=650$; Steel ratio, depth and steel area, balanced reinforcement $p=0.0077$; $d=0.028\sqrt{M}$; $A=0.0026\sqrt{M}$;
63. T-BEAMS: Neglecting compression in Web. For "t" greater than "kd," use 61. 	$k=\frac{pn+\frac{1}{2}\left(\frac{t}{d}\right)^2}{pn+\frac{t}{d}}=\frac{1}{1+\frac{f_s}{nf_c}}$; $j=1-\frac{t}{3d}\cdot\frac{3kd-2t}{2kd-t}$; $M_s=f_sAjd=f_spjbd^2$; $M_c=\left[1-\frac{t}{2kd}\right]f_ctjbd$; $f_s=\frac{M}{Ajd}=\frac{M}{pjbd^2}$; $f_c=\frac{k}{(1-k)n}f_s$; Steel ratio, balanced reinforcement, $p=\frac{t}{2d}\frac{f_c}{f_s}\left[2-\left(1+\frac{f_s}{nf_c}\right)\frac{t}{d}\right]$	$k=0.379$ $j=1-\frac{t}{3d}\,\frac{1.137d-2t}{0.758d-t}$ $M_s=16000pjbd^2$ $M_c=M_s$ $f_s=16000$; $f_c=650$; Steel ratio, balanced reinf. $p=.0203(2-2.642\frac{t}{d})\frac{t}{d}$;
64. RECTANGULAR BEAMS: Reinforced for tension and compression. 	$k=\sqrt{(p+p')^2n^2+2(p+p'r)n}-(p+p')n=\frac{1}{1+\frac{f_s}{nf_c}}$; $j=\frac{\frac{1}{2}k^2(1-\frac{1}{3}k)+(k-r)(1-r)p'n}{(1-k)pn}$; $M_s=f_sAjd=f_spjbd^2$; $M_c=\frac{1-k}{k}nf_cAjd$, $f_s=\frac{M}{Ajd}=\frac{M}{pjbd^2}$; $f_s'=\frac{k-r}{1-k}\cdot f_s$; $f_c=\frac{k}{(1-k)n}f_s$; Steel ratio, balanced reinforcement, $p=\left[\frac{nf_c}{f_s}(1-r)-r\right]p'+\frac{1}{\frac{2f_s}{f_c}\left[\frac{f_s}{nf_c}+1\right]}$;	$k=0.379$ $j=\frac{.00418+(.379-r)(1-r)p'}{.00478+(.379-r)p'}$ M_s use general formula $M_c=M_s$ $f_s=16000$; $f_s'=9750-2577r$; $f_c=650$; Steel ratio, balanced reinforcement, $p=(0.6094-1.6094r)p'+0.0077$,

65 Shear, Bond and Web Reinforcement.

In the following formulas jd refers to arm of resisting couple at section in question, and Σo, to tension bars at section.

Shear in Concrete & Bond Stress in Tensile Steel,

Rectangular Beams, (single or double reinforced) $f_v = \dfrac{V}{bjd}$; $f_u = \dfrac{V}{\Sigma ojd}$;

T-Beams, $f_v = \dfrac{V}{b'jd}$, $f_u = \dfrac{V}{\Sigma ojd}$

Stirrups, All rectangular beams and T-beams.

Vertical stirrups, $P = \dfrac{Vs}{jd}$; $s = \dfrac{Pjd}{V}$

Stirrups inclined 45°, (not bent up bars)

$$P = 0.7\frac{Vs}{jd}; \; s = \frac{Pjd}{0.7V}$$

P = Total stress in one stirrup. V = amount of shear not carried by concrete.

For approximate results $j = \frac{7}{8}$ in formulas.

66. Columns: Ratio of length to least width < 12

Axial load for given unit stress,

$$P = f_c(A_c + nA_s) = f_c A[1 + (n-1)p],$$

Unit stress for given axial load,

$$f_c = \frac{P}{A[1+(n-1)p]}; \quad f_s = nf_c,$$

67. Working Stresses for Static Loads (A.S.C.E.)

Ultimate Strengths for Various Mixtures, in Pounds per square inch

Aggregate	1:2:4	1:2½:5	1:3:6
Granite	2200	1800	1400
Gravel, Hard Limestone or sandstone	2000	1600	1300
Soft Limestone or Sandstone	1500	1200	1000

Working Stress, percent of Ultimate Strength; Bearing 32.5; Axial Comp. 22.5; Comp. Fiber Stress 32.5; Shear: longitudinal b'rs only, 2.0; Part of b'rs bent up 3.0; Shear: thorough web reinf. 6.0; Bond, brs 4.0, wire 2.0.

68. Safe Loads on Reinforced Concrete Slabs: $f_s = 16000$, $f_c = 650$, $n = 15$, $M = \frac{1}{10}wl^2$

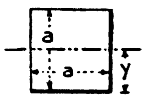
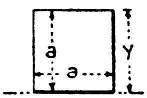
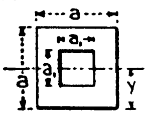
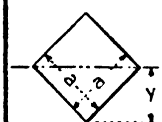
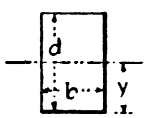
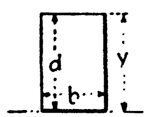
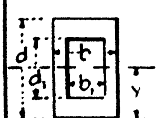
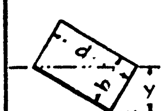
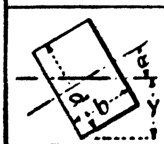
Span in Feet for Safe Live Load in Pounds per Square Foot of Slab. $M = \frac{1}{10}wl^2$ (For $M = \frac{1}{8}wl^2$ multiply span lengths by 0.894)

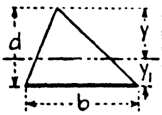
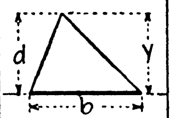
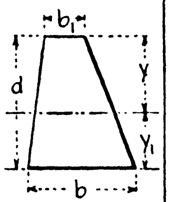
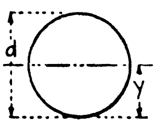
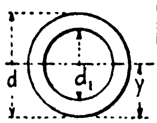
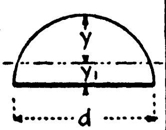
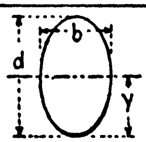
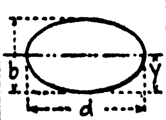
Total Thickness of Slab. In.	Center of Steel to Bottom of Slab In.	Area of Steel per ft. of width. Sq. In.	Weight of Slab per sq. ft. Lb.	40 Lb.	50 Lb.	75 Lb.	100 Lb.	125 Lb.	150 Lb.	200 Lb.	250 Lb.	300 Lb.	350 Lb.	400 Lb.
3	¾	0.208	38	8.4	7.9	7.0	6.3	5.8	5.4	4.8	4.3	4.0	3.7	3.6
3½	¾	0.254	44	9.6	9.3	8.3	7.5	6.9	6.5	5.8	5.3	4.9	4.5	4.3
4	1	0.277	50	10.4	9.0	8.8	8.0	7.4	7.0	6.2	5.7	5.3	4.9	4.7
4½	1	0.323	56	11.7	11.2	10.0	9.2	8.5	8.0	7.2	6.6	6.1	5.7	5.4
5	1	0.369	63	12.9	12.3	11.2	10.3	9.6	9.0	8.1	7.4	6.9	6.5	6.1
5½	1	0.416	69	14.1	13.5	12.3	11.3	10.6	10.0	9.0	8.3	7.7	7.2	6.8
6	1¼	0.439	75	14.5	13.9	12.7	11.8	11.0	10.4	9.4	8.6	8.0	7.5	7.1

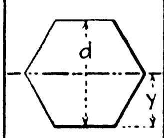
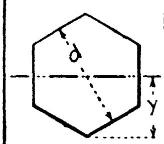
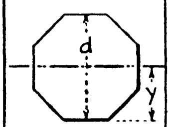
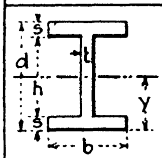
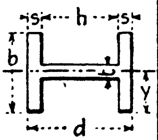
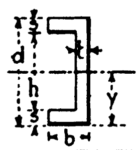
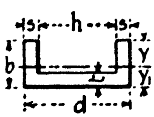
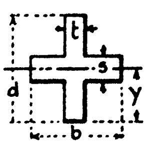
69. Safe Loads on Reinforced Concrete Slabs: $f_s = 16000$, $f_c = 650$, $n = 15$, $M = \frac{1}{12}wl^2$

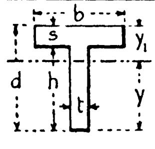
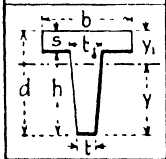
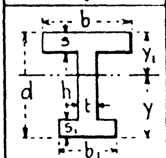
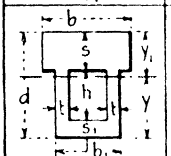
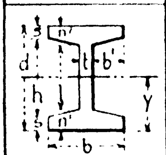
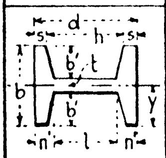
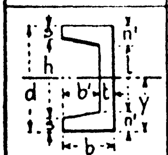
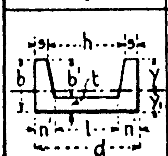
Span in Feet for Safe Live Load in Pounds per Square Foot of Slab. $M = \frac{1}{12}wl^2$, (For $M = \frac{1}{8}wl^2$ multiply span lengths by 0.817)

Total Thickness of Slab. In.	Center of Steel to Bottom of Slab. In.	Area of Steel per ft. of width. Sq. In.	Weight of Slab per sq. ft. Lb.	40 Lb.	50 Lb.	75 Lb.	100 Lb.	125 Lb.	150 Lb.	200 Lb.	250 Lb.	300 Lb.	350 Lb.	400 Lb.
3	¾	0.208	38	9.2	8.6	7.6	6.9	6.4	5.9	5.2	4.8	4.4	4.1	3.9
3½	¾	0.254	44	10.8	10.2	9.1	8.2	7.6	7.1	6.3	5.8	5.3	5.0	4.7
4	1	0.277	50	11.4	10.8	9.6	8.8	8.2	7.6	6.8	6.2	5.8	5.4	5.1
4½	1	0.323	56	12.8	12.2	11.0	10.7	9.3	8.8	7.9	7.2	6.7	6.2	5.9
5	1	0.369	63	14.2	13.5	12.2	11.3	10.5	9.9	8.9	8.1	7.5	7.1	6.7
5½	1	0.416	69	15.5	14.8	13.5	12.4	11.6	10.9	9.9	9.1	8.4	7.9	7.5
6	1¼	0.439	75	15.9	15.3	13.9	12.9	12.1	11.4	10.3	9.5	8.8	8.3	7.8

Section	Area A	Distance from Axis to Extreme Fibers y and y_1	Moment of Inertia I	Section Modulus $S=\frac{I}{y}$	Radius of Gyration $r=\sqrt{\frac{I}{A}}$
	a^2	$y=\frac{a}{2}$	$\frac{a^4}{12}$	$\frac{a^3}{6}$	$\frac{a}{\sqrt{12}}=0.289a$
	a^2	$y=a$	$\frac{a^4}{3}$	$\frac{a^3}{3}$	$\frac{a}{\sqrt{3}}=0.577a$
	$a^2-a_1^2$	$y=\frac{a}{2}$	$\frac{a^4-a_1^4}{12}$	$\frac{a^4-a_1^4}{6a}$	$\sqrt{\frac{a^2+a_1^2}{12}}$
	a^2	$y=\frac{a}{\sqrt{2}}=0.707a$	$\frac{a^4}{12}$	$\frac{a^3}{6\sqrt{2}}=0.118a^3$	$\frac{a}{\sqrt{12}}=0.289a$
	$b\cdot d$	$y=\frac{d}{2}$	$\frac{b\cdot d^3}{12}$	$\frac{b\cdot d^2}{6}$	$\frac{d}{\sqrt{12}}=0.289d$
	$b\cdot d$	$y=d$	$\frac{b\cdot d^3}{3}$	$\frac{b\cdot d^2}{3}$	$\frac{d}{\sqrt{3}}=0.577d$
	$b\cdot d-b_1d_1$	$y=\frac{d}{2}$	$\frac{b\cdot d^3-b_1d_1^3}{12}$	$\frac{b\cdot d^3-b_1d_1^3}{6\cdot d}$	$\sqrt{\frac{b\cdot d^3-b_1d_1^3}{12[b\cdot d-b_1d_1]}}$
	$b\cdot d$	$y=\frac{b\cdot d}{\sqrt{b^2+d^2}}$	$\frac{b^3d^3}{6[b^2+d^2]}$	$\frac{b^2d^2}{6\sqrt{b^2+d^2}}$	$\frac{b\cdot d}{\sqrt{6[b^2+d^2]}}$
	$b\cdot d$	$y=\frac{d\cdot\cos\alpha+b\cdot\sin\alpha}{2}$	$\frac{bd}{12}[d^2\cdot\cos^2\alpha+b^2\cdot\sin^2\alpha]$	$\frac{bd}{6}\left[\frac{d^2\cos^2\alpha+b^2\sin^2\alpha}{d\cdot\cos\alpha+b\cdot\sin\alpha}\right]$	$\sqrt{\frac{d^2\cos^2\alpha+b^2\sin^2\alpha}{12}}$

Section	Area A	Distance from Axis to Extreme Fibers y and y_1	Moment of Inertia I	Section Modulus $S=\frac{I}{y}$	Radius of Gyration $r=\sqrt{\frac{I}{A}}$
	$\frac{b\cdot d}{2}$	$y=\frac{2d}{3},\ y_1=\frac{d}{3}$	$\frac{b\ d^3}{36}$	$\frac{b\cdot d^2}{24}$	$\frac{d}{\sqrt{18}}=.236d$
	$\frac{b\cdot d}{2}$	$y=d$	$\frac{b\cdot d^3}{12}$	$\frac{b\cdot d^2}{12}$	$\frac{d}{\sqrt{6}}=.408d$
	$\frac{b+b_1}{2}\cdot d$	$y=\frac{b_1+2b}{b_1+b}\cdot\frac{d}{3}$ $y_1=\frac{b+2b_1}{b+b_1}\cdot\frac{d}{3}$	$\frac{b^2+4b\cdot b_1+b_1^2}{36(b+b_1)}\cdot d^3$	$\frac{b^2+4b\cdot b_1+b_1^2}{12(2b+b_1)}\cdot d^2$	$\frac{d}{6(b+b_1)}\sqrt{2(b^2+4b\cdot b_1+b_1^2)}$
	$\frac{\pi d^2}{4}=.785d^2$	$y=\frac{d}{2}$	$\frac{\pi d^4}{64}=.049d^4$	$\frac{\pi d^3}{32}=.098d^3$	$\frac{d}{4}$
	$\frac{\pi(d^2-d_1^2)}{4}=$ $.785(d^2-d_1^2)$	$y=\frac{d}{2}$	$\frac{\pi(d^4-d_1^4)}{64}$ $=.049(d^4-d_1^4)$	$\frac{\pi(d^4-d_1^4)}{32d}$ $=.098(d^4-d_1^4)\div d$	$\frac{\sqrt{d^2+d_1^2}}{4}$
	$\frac{\pi d^2}{8}=.393d^2$	$y=\frac{(3\pi-4)d}{6\pi}=.288d$ $y_1=\frac{2d}{3\pi}=.212d$	$\frac{9\pi^2-64}{1152\pi}\cdot d^4$ $=.007d^4$	$\frac{9\pi^2-64}{192(3\pi-4)}\cdot d^3$ $=.024d^3$	$\frac{\sqrt{9\pi^2-64}}{12\pi}\cdot d$ $=.132d$
	$\frac{\pi b\cdot d}{4}=.785b\cdot d$	$y=\frac{d}{2}$	$\frac{\pi b\cdot d^3}{64}=.049b\cdot d^3$	$\frac{\pi b\cdot d^2}{32}=.098b\cdot d^2$	$\frac{d}{4}$
	$\frac{\pi b\cdot d}{4}=.785b\cdot d$	$y=\frac{b}{2}$	$\frac{\pi d\cdot b^3}{64}=.049d\cdot b^3$	$\frac{\pi d\cdot b^2}{32}=.098d\cdot b^2$	$\frac{b}{4}$

Section	Area A	Distances to Extreme Fibers y and y_1	Moment of Inertia I	Section Modulus $S=\frac{I}{y}$	Radius of Gyration $r=\sqrt{\frac{I}{A}}$
	$\frac{3}{2}d^2\tan 30°$ $=.866d^2$	$y=\frac{d}{2}$	$\frac{A}{12}\left[\frac{d^2(1+2\cos^2 30°)}{4\cos^2 30°}\right]$ $=.06d^4$	$\frac{A}{6}\left[\frac{d(1+2\cos^2 30°)}{4\cos^2 30°}\right]$ $=.12d^3$	$\frac{d}{4}\sqrt{\frac{1+2\cos^2 30°}{3\cos^2 30°}}$ $=.264d$
	$\frac{3}{2}d^2\tan 30°$ $=.866d^2$	$y=\frac{d}{2\cdot\cos 30°}$ $=.577d$	$\frac{A}{12}\left[\frac{d^2(1+2\cos^2 30°)}{4\cdot\cos^2 30°}\right]$ $=.06d^4$	$\frac{A}{6}\left[\frac{d(1+2\cos^2 30°)}{4\cos 30°}\right]$ $=.104d^3$	$\frac{d}{4}\sqrt{\frac{1+2\cos^2 30°}{3\cos^2 30°}}$ $=.264d$
	$2d^2\tan 22\frac{1}{2}°$ $=.828d^2$	$y=\frac{d}{2}$	$\frac{A}{12}\left[\frac{d^2(1+2\cos^2 22\frac{1}{2}°)}{4\cos^2 22\frac{1}{2}°}\right]$ $=.055d^4$	$\frac{A}{6}\left[\frac{d(1+2\cos^2 22\frac{1}{2}°)}{4\cos 22\frac{1}{2}°}\right]$ $=.109d^3$	$\frac{d}{4}\sqrt{\frac{1+2\cos^2 22\frac{1}{2}°}{3\cos^2 22\frac{1}{2}°}}$ $=.257d$
	$b\cdot d-h(b-t)$	$y=\frac{d}{2}$	$\frac{b\cdot d^3-h^3(b-t)}{12}$	$\frac{b\cdot d^3-h^3(b-t)}{6d}$	$\sqrt{\frac{b\cdot d^3-h^3(b-t)}{12[b\cdot d-h(b-t)]}}$
	$b\cdot d-h(b-t)$	$y=\frac{b}{2}$	$\frac{2s\cdot b^3+h\cdot t^3}{12}$	$\frac{2s\cdot b^3+h\cdot t^3}{6b}$	$\sqrt{\frac{2s\cdot b^3+h\cdot t^3}{12[b\cdot d-h(b-t)]}}$
	$b\cdot d-h(b-t)$	$y=\frac{d}{2}$	$\frac{b\cdot d^3-h^3(b-t)}{12}$	$\frac{b\cdot d^3-h^3(b-t)}{6d}$	$\sqrt{\frac{b\cdot d^3-h^3(b-t)}{12[b\cdot d-h(b-t)]}}$
	$b\cdot d-h(b-t)$	$y=\frac{\frac{1}{2}[b^2d-h(b-t)^2]}{b\cdot d-h(b-t)}$ $y_1=b-y$	$\frac{2b^3s+ht^3}{3}-Ay_1^2$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
	$t\cdot d+s(b-t)$	$y=\frac{d}{2}$	$\frac{t\cdot d^3+s^3(b-t)}{12}$	$\frac{t\cdot d^3+s^3(b-t)}{6d}$	$\sqrt{\frac{td^3+s^3(b-t)}{12[td+s(b-t)]}}$

Section	Area A	Distances from Axis to Extreme Fibers y and y_1	Moment of Inertia I	Sec. Modulus $S=\frac{I}{y}$	Radius of Gyration r
	$bs+ht$	$y_1=\frac{d^2t+s^2(b-t)}{2A}$ $y=d-y_1$	$\frac{ty^3+by_1^3-(b-t)(y_1-s)^3}{3}$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
	$bs+\frac{h}{2}(t+t_1)$	$y_1=\frac{3bs^2+3th(d+s)+h(t_1-t)(h+3s)}{6A}$ $y=d-y_1$	$\frac{4bs^3+h^3(3t+t_1)}{12}-A(y_1-s)^2$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
	$bs+ht+b_1s_1$	$y=\frac{td^2+[b_1-t]s_1^2+[b-t][2d-s]s}{2A}$ $y_1=d-y$	$\frac{b_1y^3+by_1^3-[b_1-t][y-s_1]^3-[b-t][y_1-s]^3}{3}$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
	$bs+2ht+b_1s_1$	$y=\frac{2td^2+[b_1-2t]s_1^2+[b-2t][2d-s]s}{2A}$ $y_1=d-y$	$\frac{b_1y^3+by_1^3-[b_1-2t][y-s_1]^3-[b-2t][y_1-s]^3}{3}$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
	$td+2b'(s+n')$	$y=\frac{d}{2}$ g = slope of flange $=(n'-s)\div b'=(h-l)\div(b-t)=\frac{1}{6}$ for standard sections	$\frac{1}{12}\left[bd^3-\frac{1}{4g}(h^4-l^4)\right]$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
	$td+2b'(s+n')$	$y=\frac{b}{2}$ g = slope of flange $=(n'-s)\div b'=(h-l)\div(b-t)=\frac{1}{6}$ for standard sections	$\frac{1}{12}\left[b^3(d-h)+lt^3+\frac{g}{4}(b^4-t^4)\right]$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
	$td+b'(s+n')$	$y=\frac{d}{2}$ g = slope of flange $=(n'-s)\div b'=(h-l)\div 2(b-t)=\frac{1}{6}$ for standard sections	$\frac{1}{12}\left[bd^3-\frac{1}{8g}(h^4-l^4)\right]$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$
	$td+b'(s+n')$	$y_1=\left[b^2s+\frac{ht^2}{2}+\frac{g}{3}(b-t)^2(b+2t)\right]\div A$ $y=b-y_1$ g = slope of flange $=(n'-s)\div b'=(h-l)\div 2(b-t)=\frac{1}{6}$ for standard sec.	$\frac{1}{3}\left[2sb^3+lt^3+\frac{g}{2}(b^4-t^4)\right]-Ay_1^2$	$\frac{I}{y}$	$\sqrt{\frac{I}{A}}$

STRESSES IN FRAMED STRUCTURES.

Loads.—The stresses in roof trusses are due to (1) the dead load, (2) the snow load, (3) the wind load, and (4) concentrated and moving loads. Data for dead loads, snow loads, wind loads, crane loads and other loads to be carried on trusses are given in Chapter I to Chapter IV, inclusive. The loads on roof trusses are commonly given as a certain number of lb. per sq. ft. of horizontal projection of the roof. The loads are assumed to be transferred to the truss by means of purlins acting as simple beams, the joint loads being equal to the purlin reactions.

Methods of Calculation.—The determination of the reactions of simple framed structures usually requires the use of the three fundamental equations of equilibrium

$$\Sigma \text{ horizontal components of forces} = 0 \qquad (a)$$

$$\Sigma \text{ vertical components of forces} = 0 \qquad (b)$$

$$\Sigma \text{ moments of forces about any point} = 0 \qquad (c)$$

Having completely determined the external forces, the internal stresses may be obtained by either equations (*a*) and (*b*) (resolution), or equation (*c*) (moments). These equations may be solved by graphics or by algebra. There are, therefore, four methods of calculating stresses:

Resolution of Forces $\begin{cases} \text{Graphic Method} \\ \text{Algebraic Method} \end{cases}$

Moments of Forces $\begin{cases} \text{Graphic Method} \\ \text{Algebraic Method} \end{cases}$

The stresses in any simple framed structure can be calculated by using any one of the four methods. The method of calculating the stresses in roof trusses by means of graphic resolution will be explained in detail. For the calculation of the stresses in roof trusses and other framed structures by algebraic resolution and by algebraic and graphic moments the reader is referred to the author's "The Design of Steel Mill Buildings."

Graphic Resolution.—In Fig. 1 the reactions R_1 and R_2 are found by means of the force and equilibrium polygons as shown in (*b*) and (*a*). The principle of the force polygon is then applied to each joint of the structure in turn. Beginning at the joint L_0, the forces are shown in (*c*), and the force triangle in (*d*). The reaction R_1 is known and acts up, the upper chord stress $1-x$ acts downward to the left, and the lower chord stress $1-y$ acts to the right, closing the polygon. Stress $1-x$ is compression and stress $1-y$ is tension, as can be seen by applying the arrows to the members in (*c*). The force polygon at joint U_1 is then constructed as in (*f*). Stress $1-x$ acting toward joint U_1 and load P_1 acting downward are known, and stresses $1-2$ and $2-x$ are found by completing the polygon. Stresses $2-x$ and $1-2$ are compression. The force polygons at joints L_1 and U_2 are constructed, in the order given, in the same manner. The known forces at any joint are indicated in direction in the force polygon by double arrows, and the unknown forces are indicated in direction by single arrows.

The stresses in the members of the right segment of the truss are the same as in the left, and the force polygons are, therefore, not constructed for the right segment. The force polygons for all the joints of the truss are grouped into the stress diagram shown in (*k*). Compression in the stress diagram and truss is indicated by arrows acting toward the ends of the stress lines and toward the joints, respectively, and tension is indicated by arrows acting away from the ends of the stress lines and away from the joints, respectively The first time a stress is used a single arrow, and the second time the stress is used a double arrow is used to indicate direction. The stress diagram in (*k*) Fig. 1 is called a Maxwell diagram or a reciprocal polygon diagram, *i. e.*, areas in the truss diagram become points in the stress diagram. The notation used is known as Bow's notation. The method of graphic resolution is the method most commonly used for calculating stresses in roof trusses and in simple framed structures with inclined chords.

STRESSES IN ROOF TRUSSES.—The methods of calculating dead load, snow load, and wind load stresses in roof trusses by graphic resolution will be briefly described.

Dead Load Stresses.—The dead load is made up of the weight of the truss and the roof covering, and is usually considered as applied at the panel points of the upper chords in computing stresses in roof trusses. If the purlins do not come at the panel points, the upper chord will have to be designed for direct stress and stress due to flexure.

The stress in a Fink truss due to dead loads is calculated by graphic resolution in (*a*) Fig. 2.

The loads are laid off, the reactions found, and the stresses calculated beginning at joint L_0, as explained in Fig. 1. The stress diagram for the right half of the truss need not be drawn where the truss and loads are symmetrical as in (*a*) Fig. 2; however, it gives a check on the accuracy of the work and is well worth the extra time required. The loads P_1 on the abutments have no effect on the stresses in the truss, and may be omitted in this solution.

In calculating the stresses at joint P_3, the stresses in the members 3–4, 4–5 and *x*–5 are unknown, and the solution appears to be indeterminate. The solution is easily made by cutting out members 4–5 and 5–6, and replacing them with the dotted member shown. The stresses in the members in the modified truss are now obtained up to and including stresses 6–*x* and 6–7. Since the stresses 6–*x* and 6–7 are independent of the form of the framework to the left, as can easily be seen by cutting a section through the members 6–*x*, 6–7 and 7–*y*, the solution can be carried back and the apparent ambiguity removed. The ambiguity can also be removed by calculating the stress in 7–*y* by algebraic moments and substituting it in the stress diagram. It will be noted that all top chord members are in compression and all bottom chord members are in tension.

Snow Load Stresses.—Large snow storms nearly always occur in still weather, and the maximum snow load will therefore be a uniformly distributed load. A heavy wind may follow a sleet storm and a snow load equal to the minimum given in § 20, "Specifications for Steel Frame Buildings," Chapter I, should be considered as acting at the same time as the wind load. The stresses due to snow load are found in the same manner as the dead load stresses.

Wind Load Stresses.—The stresses in trusses due to wind load will depend upon the direction and intensity of the wind, and the condition of the end supports. The wind is commonly considered as acting horizontally, and the normal component, as determined by one of the formulas in § 21, "Specifications for Steel Frame Buildings," Chapter I, is taken.

The ends of the truss may (1) be rigidly fixed to the abutment walls, (2) be equally free to move, or (3) may have one end fixed and the other end on rollers. When both ends of the truss are rigidly fixed to the abutment walls (1) the reactions are parallel to each other and to the resultant of the external loads; where both ends of the truss are equally free to move (2) the horizontal components of the reactions are equal; and where one end is fixed and the other end is on frictionless rollers (3) the reaction at the roller end will always be vertical. Either case (1) or case (3) is commonly assumed in calculating wind load stresses in trusses. Case (2) is the condition in a portal or a framed bent. The vertical components of the reactions are independent of the condition of the ends.

Wind Load Stresses: No Rollers.—The stresses due to a normal wind load, in a Fink truss with both ends fixed to rigid walls, are calculated by graphic resolution in (*b*) Fig. 2. The reactions are parallel and their sum equals the sum of the external loads; they are found by means of force and equilibrium polygons. To calculate the reactions, lay off the loads P_1, P_2, P_3, P_4, P_5, as shown, and select the pole O at any convenient point. Then at a point on line of action of P_1 in the truss diagram, draw strings parallel to the rays drawn through the ends of P_1 in the force polygon. The string drawn parallel to the ray common to forces P_1 and P_2 in the force polygon will cut the force P_2 in the truss diagram. Through this point draw a string parallel to the ray common to forces P_2 and P_3 in the force polygon, and so on until the strings drawn parallel to the outside rays meet on the resultant of all the loads. The closing line of the force polygon connects the two points on the reactions. Through point O in the force polygon draw line O–Y parallel to the closing line in the equilibrium polygon, R_1 and R_2 are the reactions, as shown.

The stress diagram is constructed in the same manner as that for dead loads. Heavy lines in truss and stress diagram indicate compression and light lines indicate tension.

The ambiguity at joint P_2 is removed by means of the dotted member, as in the case of the dead load stress diagram. It will be seen that there are no stresses in the dotted web members in the right segment of the truss. It is necessary to carry the solution entirely through the truss, beginning at the left reaction and checking up at the right reaction. It will be seen that the load P_1 has no effect on the stresses in the truss in this case, the left reaction being simply reduced if P_1 is omitted.

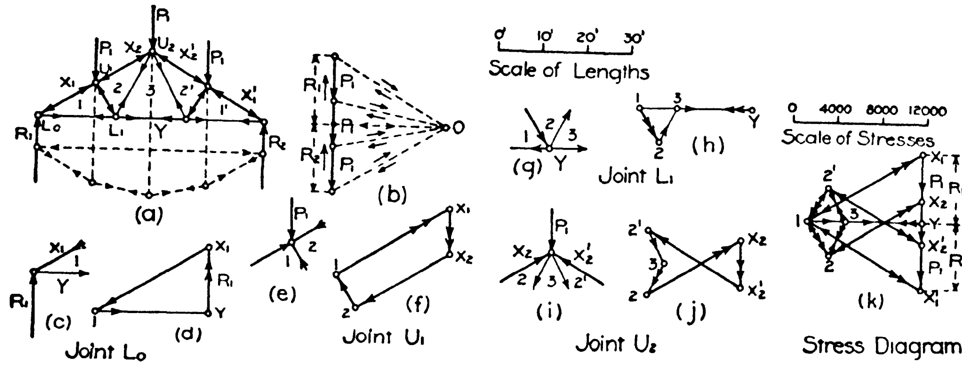


FIG. 1.

Wind Load Stresses: Rollers.—Trusses longer than 70 ft. are usually fixed at one end, and are supported on rollers at the other end. The reaction at the roller end is then vertical—the horizontal component of the external wind force being all taken by the fixed end. The wind may come on either side of the truss, giving rise to two conditions: (1) rollers leeward and (2) rollers windward, each requiring a separate solution.

Rollers Leeward.—The wind load stresses in a triangular Pratt truss with rollers under the leeward side are calculated by graphic resolution in (*c*) Fig. 2.

The reactions in (*c*) Fig. 2 were first determined by means of force and equilibrium polygons, on the assumption that they were parallel to each other and to the resultant of the external loads. Then since the reaction at the roller end is vertical and the horizontal component at the fixed end is equal to the horizontal component of the external wind forces, the true reactions were obtained by closing the force polygon.

In order that the truss be in equilibrium under the action of the three external forces, R_1, R_2 and the *resultant of the wind loads*, the three external forces must meet in a point if produced. This furnishes a method for determining the reactions, where the direction and line of action of one and a point in the line of action of the other are known, providing the point of intersection of the three forces comes within the limits of the drawing board.

The stress diagram is constructed in the same way as the stress diagram for dead loads. It will be seen that the load P_1 has no effect on the stresses in the truss in this case. Heavy lines in truss and stress diagram indicate compression, and light lines indicate tension.

Rollers Windward.—The wind load stresses in the same triangular Pratt truss as shown in (*c*) Fig. 2, with rollers under the windward side of the truss are calculated by graphic resolution in (*d*) Fig. 2.

The true reactions were determined directly by means of force and equilibrium polygons. The direction of the reaction R_1 is known to be vertical, but the direction of the reaction R_2 is unknown, the only known point in its line of action being the right abutment. The equilibrium polygon is drawn to pass through the right abutment and the direction of the right reaction is determined by connecting the point of intersection of the vertical reaction R_1 and the line drawn through O parallel to the closing line of the equilibrium polygon, with the lower end of the load line.

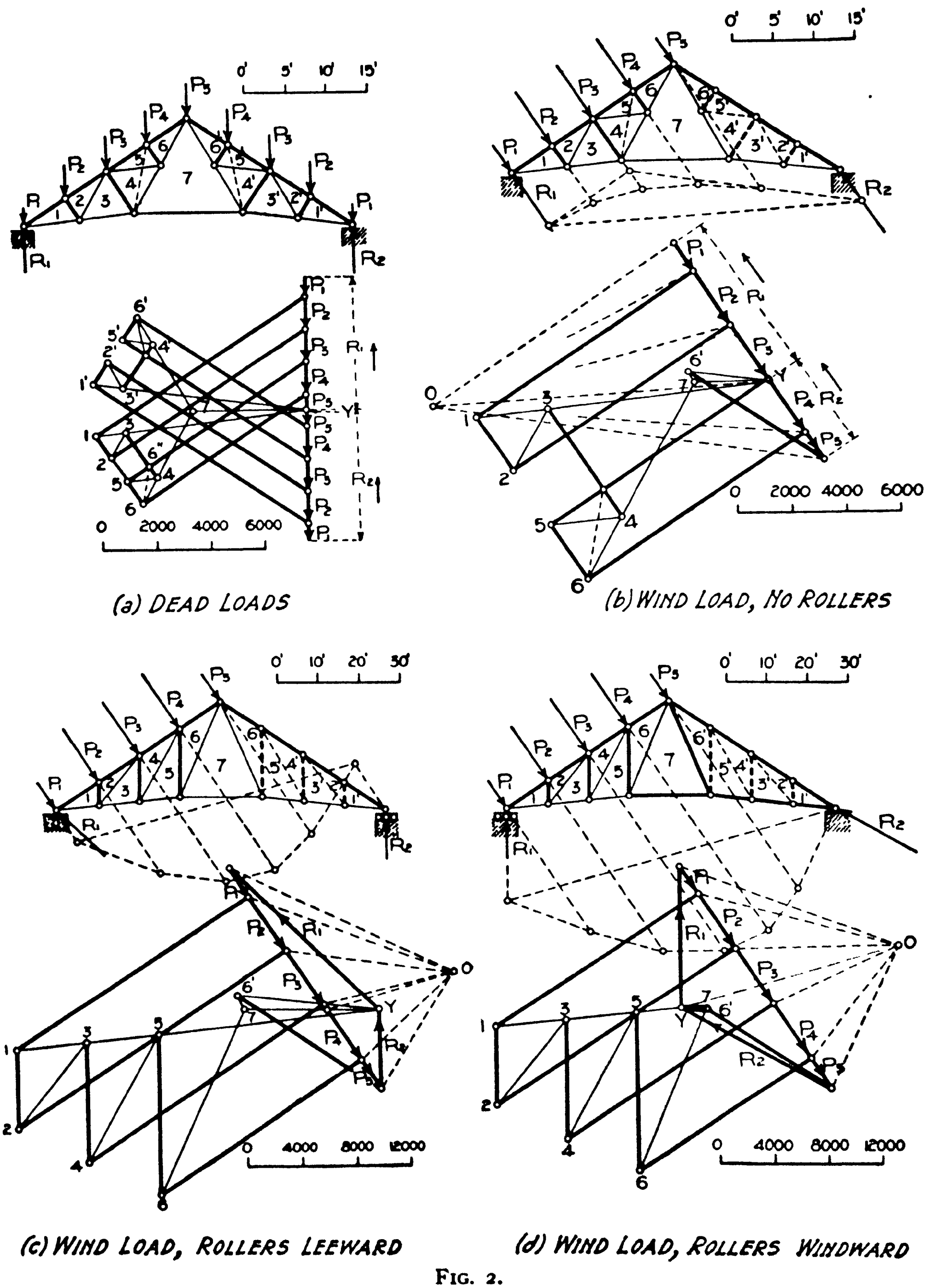

(a) DEAD LOADS
(b) WIND LOAD, NO ROLLERS
(c) WIND LOAD, ROLLERS LEEWARD
(d) WIND LOAD, ROLLERS WINDWARD

FIG. 2.

Since the vertical components of the reactions are independent of the conditions of the ends of the truss, the vertical components of the reactions in (*c*) and (*d*) Fig. 2 are the same. It will be seen that the load P_1 produces stress in the members of the truss with rollers windward. If the line of action of R_2 drops below the joint P_5, the lower chord of the truss will be in compression, as will be seen by taking moments about P_5.

STRESSES IN A TRANSVERSE BENT.—A transverse bent in a steel mill building consists of a roof truss supported at the ends on columns and braced against longitudinal movement by means of knee braces, Fig. 3. The ends of the columns may be fixed at the base or may be free to turn (pin-connected). The stresses in a transverse bent are statically indeterminate and cannot be calculated without taking in account the deformations of the members themselves. The following approximate method, proposed by the author in the first edition of "The Design of Steel Mill Buildings," 1903, gives results that are approximately correct, are on the safe side, and is the method now used in practice.

Dead and Snow Load Stresses.—The stresses due to dead and snow loads in trusses of a transverse bent are calculated the same as though the trusses were supported on solid walls.

Wind Load Stresses.—The external wind loads may be taken (1) as horizontal or (2) as normal to the surface. The columns will be assumed to be pin-connected at the tops and to be either pin-connected or fixed at the base. It will be assumed that the horizontal reactions at the foot of the columns are equal to each other, and equal to one-half of the horizontal component of the external wind load. It is also assumed that the truss does not change its length, and that the deflection of the columns at the top of the columns and at the foot of the knee brace are equal.

It is shown in "The Design of Steel Mill Buildings" that when the columns are fixed at the base the point of contra-flexure comes at a distance of from ½ to ⅝ of the distance from the foot of the column to the foot of the knee brace. It is usually assumed that the point of contra-flexure is located at a point in the column one-half the distance from the foot of the column to the foot of the knee brace. If h = height of the column, d = height from the base of the column to the foot of the knee brace, then the distance from the base of the column to the point of contra-flexure will be

$$y_0 = \frac{d}{2}\frac{(d + 2h)}{(2d + h)}. \tag{4}$$

The calculation of the wind stresses in a transverse bent with a monitor ventilator is shown in Fig. 3. The bents are spaced 32 ft. centers and are designed for a horizontal wind load of 20 lb. per sq. ft., the normal wind load being calculated by Hutton's formula, Fig. 3, Chapter I. The point of contra-flexure is found by substituting in equation (4) to be

$$y_0 = \frac{30.5}{2}\left(\frac{30.5 + 85}{61 + 42.5}\right) = 17 \text{ ft.}$$

The external forces are calculated for the bent above the point of contra-flexure by multiplying the area supported at the point by the intensity of the wind pressure. For example, the load at B is $32' \times 6.75' \times 20$ lb. = 4320 lb.

The line of application and the amount of the external wind load, ΣW, is found by means of a force and an equilibrium polygon. ΣW acts through the intersection of the strings parallel to the rays O–B and O–C, and is equal to C–B (line C–B is not drawn in force polygon) in amount. The reactions R and R' may be calculated graphically as follows:—Lay off the total wind load ΣW so that it will be bisected by point A in Fig. 3. Perpendiculars dropped from the ends of load line ΣW to the dotted lines AB and AC will give V' = 12,800 lb., and V = 700 lb., respectively. Then R and R' are calculated as shown.

The calculation of stresses is begun at point B in the windward column, and in the stress diagram the stresses at B are found by drawing the force polygon a–B–A–b–a. The remaining stresses are calculated as for a simple truss. In calculating the stresses in the ventilator it was assumed that diagonals 9–10 and 10–12 are tension members, so that 9–10 will not be in action

when the wind is acting as shown. Before solving the stresses at the joint 6–7–9 it was necessary to calculate the stresses in members *i*–11, 10–11 and 9–*h*. The remainder of the solution offers no difficulty to one familiar with the principles of graphic statics.

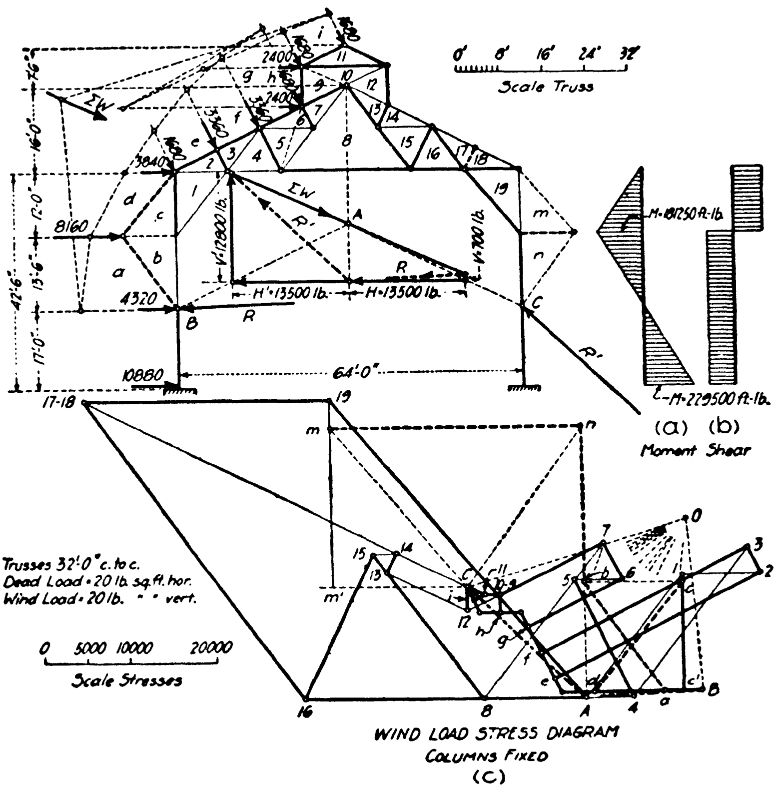


FIG. 3.

The stress in post *b*–*a* is equal to *V*, while the stress in 1–*c* is found by extending 1–*c* to *c*′ in the stress diagram, *c*′ being a point on the load line. The stress in post *n*–*A* is equal to *V*″, while the stress in 19–*m* is found by extending 19–*m* to *m*′ in the stress diagram, *m*′ being a point on the horizontal line drawn through *C*. The kind of stress in the different members is shown by the weight of lines in the bent and stress diagrams.

For a detailed discussion of the calculations of the stresses in a transverse bent, see " The Design of Steel Mill Buildings."

STRESSES IN BRIDGE TRUSSES.—The stresses in bridge trusses may be calculated by applying the condition equations for equilibrium for translation, resolution; or by applying the condition equation for equilibrium for rotation, moments. Both resolution and moments may be calculated algebraically or graphically, giving four methods for calculation the same as for roof trusses.

Maximum Stresses.—The criteria for loading a truss or beam for maximum and minimum stresses are given on page 160, Chapter IV.

Problems.—The methods of calculating the stresses in bridge trusses are shown by several problems taken from the author's " The Design of Highway Bridges."

PROBLEM 1. DEAD LOAD STRESSES IN A CAMEL-BACK TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Camel-back (inclined Pratt) truss, span 160′ 0″, panel length 20′ 0″, depth at the hip 25′ 0″, depth at the center 32′ 0″, dead load 400 lb. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 10,000 lb.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate the stresses by graphic resolution, beginning at R_1 and checking up at R_2. Follow the order given in the stress diagram.

(*c*) **Results.**—The top chord is in compression and the bottom chord is in tension. All inclined web members are in tension; while part of the posts are in compression and part are in tension. Member 1–2 is simply a hanger and is always in tension

PROBLEM 2. DEAD LOAD STRESSES IN A PETIT TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Petit truss, span 350′ 0″, panel length 25′ 0,″ depth at hip 50′ 0″, depth at center 58′ 0″, dead load 0.9 tons per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 50′ 0″. Scale of loads, 1″ = 45 tons.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the top downwards. Calculate R_1 and R_2. Calculate the stresses in the members at the left reaction by constructing force triangle 1–Y–X. Then calculate the stress in 1–2 by constructing polygon Y–1–2–Y. Draw 3–2, which is the stress in member 3–2. Then pass to joint W_2 where there appears to be an ambiguity, stress 4–5 being unknown. To remove the ambiguity proceed as follows: At W_3 on the left side of the stress diagram assume that W_3 is the stress in 5–6 (the member 5–6 is simply a hanger and the stress is as assumed). Calculate the stress in 4–5 by completing the triangle of stresses in the auxiliary members. The stresses are now all known at W_2 except 3–4 and 5–Y, but the stress in 4–5 is between the two unknown stresses. First complete the force polygon 2–3–4–5′–Y–Y–2. Then by changing the order the true polygon 2–3–4–5–Y–Y–2 may be drawn. This solution is sometimes called the method of sliding in a member. The apparent ambiguity at joint W_4 may be removed in the same manner. The stress diagram is carried through as shown and finally checked up at R_2. It will be seen that there is no apparent ambiguity on the right side of the truss.

(*c*) **Results.**—It will be seen that the Petit truss is an inclined Pratt or Camel-back truss with subdivided panels. The auxiliary members are commonly tension members in all except the end primary panels as in the Baltimore truss in Problem 6. It will be seen that the stresses in the first four panels of the lower chord are the same. The loads in this type of Petit truss are carried directly to the abutments. The Petit truss is quite generally used for long span highway and railway bridges.

PROBLEM 3. MAXIMUM AND MINIMUM STRESSES IN A WARREN TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Warren truss, span 160′ 0″, panel length 20′ 0″, depth 20′ 0″, dead load 800 lb. per lineal foot per truss, live load 1,600 lb. per lineal foot per truss. Calculate the maximum and minimum stresses in the members due to dead and live loads by algebraic resolution. Scale of truss as shown.

(*b*) **Methods.—Dead Load Stresses.**—Beginning at the left end the left reaction is $R_1 = 3\frac{1}{2}W$ The shear in the first panel is $3\frac{1}{2}W$, in the second panel is $2\frac{1}{2}W$, in the third panel is $\frac{3}{2}W$, and in the fourth panel is $\frac{1}{2}W$. Now resolving at R_1 the stress in 1–Y = $-3\frac{1}{2}W\cdot\tan\theta$, stress 1–$X$ = $+3\frac{1}{2}W\cdot\sec\theta$. Cut members 1–$Y$, 1–2 and 2–$X$ and the truss to the right by a plane and equate the horizontal components of the stresses in the members. The unknown stress 2–X will equal the sum of the horizontal components of the stresses in 1–Y and 1–2 with sign changed, $= -(-3\frac{1}{2} - 3\frac{1}{2})W\cdot\tan\theta = +7W\tan\theta$. The stress in 3–$Y$ $= -(7 + 2\frac{1}{2})W\tan\theta = -9\frac{1}{2}W\cdot\tan\theta$. Stress in 4–$X$ $= -(-9\frac{1}{2} - 2\frac{1}{2})W\cdot\tan\theta = +12W\cdot\tan\theta$; stress in 5–$Y$ $= -(+12 + 1\frac{1}{2})W\cdot\tan\theta = +13\frac{1}{2}W\cdot\tan\theta$; and the stress in 6–$X$ $= -(-13\frac{1}{2} - 1\frac{1}{2})W\cdot\tan\theta = +15W\cdot\tan\theta$; etc. The coefficients of the chord stresses when multiplied by $W\tan\theta$ give the stresses, while the coefficients for the webs when multiplied by $W\cdot\sec\theta$ give the web stresses.

Live Load Stresses.—*Chord Stresses*—The maximum chord stresses occur when the joints are all loaded, and the chord coefficients are found as for dead loads. The minimum live load stresses in the chords occur when none of the joints are loaded, and are zero for each member.

Web Stresses.—The maximum web stresses in any panel occur when the longer segment into which the panel divides the truss is loaded, while the shorter segment has no loads on it. The minimum live load web stresses occur when the shorter segment is loaded and the longer segment has no loads on it. The maximum stresses in members 1–X and 1–2 occur when the truss is fully

loaded. The shear in the panel is $3\frac{1}{2}P$, or $\frac{28}{8}P$, and the stress in $1\text{-}X = 3\frac{1}{2}P \cdot \sec\theta = +125{,}400$ lb., while the stress in $1\text{-}2 = -3\frac{1}{2}P \cdot \sec\theta = -125{,}400$ lb. The minimum stresses in $1\text{-}X$ and 1–2 are zero. The maximum stresses in 2–3 and 3–4 occur when 6 loads are on the right of the panel and there are no loads on the left of the panel. The shear in the panel will then be equal to the left reaction, $= R_1 = (6 \times 3\frac{1}{2} \times P)/8 = \frac{21}{8}P$. The stress in $2\text{-}3 = \frac{21}{8}P \cdot \sec\theta = +94{,}080$ lb., while the stress in $3\text{-}4 = -\frac{21}{8}P \cdot \sec\theta = -94{,}080$ lb. The minimum stresses in 2–3 and 3–4 will occur when there is one load on the shorter segment. In the corresponding panel on the right of the truss, if the shorter segment is loaded, the left reaction $= \frac{1}{8}P =$ the shear in the panel. The minimum stress in $2\text{-}3 = -\frac{1}{8}P \cdot \sec\theta = -4{,}480$ lb., while the minimum stress in $3\text{-}4 = +4{,}480$ lb. The stresses in the remaining panels are calculated in the same manner. The maximum chord stresses are equal to the sum of the dead and live load chord stresses. The minimum chord stresses are the dead load chord stresses. The maximum web stresses are equal to the sum of the dead and the maximum live load web stresses. The minimum web stresses are equal to the algebraic sum of the dead load stresses and the minimum live load stresses.

(*c*) **Results.**—The web members 7–6 and 7–8 have a reversal of stress from tension to compression, or the reverse. These members must be counterbraced to take both kinds of stress.

Problem 4. Maximum and Minimum Stresses in a Pratt Truss by Algebraic Resolution.

(*a*) **Problem.**—Given a Pratt truss, span 140′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 800 lb. per lineal foot per truss, live load 1,600 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 20′ 0″.

(*b*) **Methods.**—Construct three truss diagrams as shown. On the first place the dead load coefficients and the dead load stresses. On the second place the live load coefficients and the live load stresses. On the third place the maximum and minimum stresses due to dead and live loads. The maximum chord stresses are the sums of the dead and live load chord stresses, while the minimum chord stresses are those due to dead load alone. The hip vertical is simply a hanger and has a minimum stress of one dead load and a maximum stress of one live and one dead load. The conditions for maximum and minimum stresses in the webs are the same as for the Warren truss, the vertical posts having stresses equal to the vertical components of the stresses in the inclined web members meeting them on the unloaded (top) chord.

(*c*) **Results.**—There is no dead load shear in the middle panel, but it is seen that there are stresses in the counters for live loads. Only one of the counters will be in action at one time Whenever the center of gravity of the loads is not in the center line of the truss, that counter will be acting that extends downward toward the center of gravity. The numerators of the maximum and minimum live load web coefficients are 0, 1, 3, 6, 10, 15, 21, as for the Warren truss. This shows that the maximum and minimum web stresses are proportional to the ordinates to a parabola.

Problem 5. Maximum and Minimum Stresses in a Deck Baltimore Truss by Algebraic Resolution.

(*a*) **Problem.**—Given a deck Baltimore truss, span 280′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 0.375 tons per lineal foot per truss, live load 0.625 tons per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution.

(*b*) **Methods.**—Construct three truss diagrams and use them as shown.

Dead Load Stresses.—The auxiliary struts 1–2, 5–6, 9–10, etc., carry a full dead load compression, while the auxiliary web members 2–3, 6–7, 10–11, etc., have a tensile stress of $\frac{1}{2}W \cdot \sec\theta$. The stress in $1\text{-}Y$ equals the shear in the panel multiplied by $\sec\theta = -6\frac{1}{2}W \cdot \sec\theta$. The stress in $3\text{-}Y$ equals the shear in the panel multiplied by $\sec\theta$, plus the inclined component of the one-half load that is carried toward the center by the auxiliary member 2-3, $= -(5\frac{1}{2} + \frac{1}{2})W \cdot \sec\theta = -6W \cdot \sec\theta$. The stress in 3–4 is the vertical component of the stress in $3\text{-}Y = +6W$. The stress in $4\text{-}Y$ is the horizontal component of the stress in $3\text{-}Y = -6W \cdot \tan\theta$. The stress in $1\text{-}X$ and $2\text{-}X = +6\frac{1}{2}W \cdot \tan\theta$. The stress in 4–5 is the inclined component of the shear in the panel $= -4\frac{1}{2}W \cdot \sec\theta$. The stress in $5\text{-}X = -(-6 - 4\frac{1}{2})W \cdot \tan\theta = +10\frac{1}{2}W \cdot \tan\theta$. The remaining dead load stresses are calculated in a similar manner.

Live Load Web Stresses.—The maximum shears in the different panels occur when the longer segment of the truss is loaded, while the minimum shears occur when the shorter segment of the truss is loaded. The maximum stresses in the webs in the first and second panels occur for a full live load on the bridge. The maximum shear in the third panel occurs with all loads to the right of the panel and no loads to the left. The shear in the panel will then be equal to the left reaction $= 11 \times \frac{1}{2}(11 + 1)P/14 = \frac{33}{7}P$. The maximum live load stress in 4–5 will be =

$- \frac{59}{14} P \cdot \sec\theta$. With a maximum stress in 4–5 the stress in 4–7 will be $= (-66/14 + 7/14)P \cdot \sec\theta = -\frac{59}{14}P \cdot \sec\theta$. This is the maximum stress, for the stress in 4–7 when there is a maximum shear in the panel is $= 10 \times 11/2 \times \frac{1}{14}P \cdot \sec\theta = -\frac{55}{14}P \cdot \sec\theta$. In a similar manner it will be found that maximum stresses in members 8–9 and 8–11 occur with a maximum shear in 8–9. On the right side it will be seen that minimum stresses in the diagonals occur for a minimum shear in the odd-numbered panels from the right.

(*c*) **Results.**—The dead and live loads were assumed as applied on the upper chord. The upper chords are in compression, while the lower chords are in tension the same as for a through truss. The live and dead load stresses are given separately on the left side of the lower truss.

PROBLEM 6. MAXIMUM AND MINIMUM STRESSES IN A THROUGH BALTIMORE TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a through Baltimore truss, span 320′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 800 lb. per lineal foot per truss, live load 1,800 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 40′ 0″.

(*b*) **Methods.**—Construct three truss diagrams as shown.

Dead Load Stresses.—The shear in each of the hangers is W, while the stress in each of the diagonal auxiliary members is $-\frac{1}{2}W \cdot \sec\theta$. The stress in the upper part of the end-post is $(+6\frac{1}{2} + \frac{1}{2})W \cdot \sec\theta = +7W \cdot \sec\theta$, where $+6\frac{1}{2}W \cdot \sec\theta$ is the stress due to the shear and $+\frac{1}{2}W \cdot \sec\theta$ is the stress due to the half load carried toward the center by the auxiliary diagonal member. The stress in the main diagonal in the third panel is $-5\frac{1}{2}W \cdot \sec\theta$, where $5\frac{1}{2}W$ is the shear in the panel; while the stress in the diagonal in the fourth panel is $(-4\frac{1}{2} - \frac{1}{2})W \cdot \sec\theta = -5W \cdot \sec\theta$, where $4\frac{1}{2}W \cdot \sec\theta$ is the stress due to the shear in the panel and $\frac{1}{2}W \cdot \sec\theta$ is the stress carried toward the center of the truss by the auxiliary member. The chord coefficients are calculated as in Problem 5.

Live Load Stresses.—The maximum shear in the third panel occurs with 13 loads to the right of the panel and with no loads to the left of the panel. The shear in the panel is then equal to the left reaction, equals $13 \times \frac{1}{2}(13 + 1) \times P/16 = \frac{91}{16}P$. The stress in the main diagonal in the third panel is then equal to $-\frac{91}{16}P \cdot \sec\theta$. The stress in the main diagonal in the fourth panel is $(-\frac{91}{16}P + \frac{8}{16}P)\sec\theta = -\frac{83}{16}P \sec\theta$, = a maximum, the maximum shear in the panel being $12 \times \frac{1}{2}(12 + 1) \times P/16 = \frac{78}{16}P$. In like manner the maximum stresses are found in 5th and 6th panels when there is a maximum shear in the 5th panel, and in the 7th and 8th panels when there is a maximum shear in the 7th panel. Minimum stresses in the 3d and 4th panels from the right abutment occur when there is a minimum shear in the 3d panel; and in the 5th and 6th panels when there is a minimum shear in the 5th panel.

(*c*) **Results.**—The double panels next to the center require counters. It should be noticed that in calculating the stresses in these counters the diagonal auxiliary ties will have the dead load stress of + 5.66 tons as a minimum.

PROBLEM 7. MAXIMUM AND MINIMUM STRESSES IN A CAMEL-BACK TRUSS BY ALGEBRAIC MOMENTS.

(*a*) **Problem.**—Given a Camel-back truss, span 100′ 0″, panel length 20′ 0″, depth at hip 20′ 0″, depth at center 25′ 0″, dead load 300 lb. per lineal foot per truss, live load 800 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic moments. Scale of truss, 1″ = 20′ 0″.

(*b*) **Methods.**—Calculate the arms of the forces as shown and check the values by scaling from the drawing.

Dead Load Stresses.—To calculate the stress in the end-post L_0U_1, take center of moments at L_1, and pass a section cutting L_0U_1, U_1L_1 and L_1L_2, and cutting away the truss to the right. Then assume stress L_0U_1 as an external force acting from the outside toward the cut section, and stress $L_0U_1 \times 14.14 - R_1 \times 20 = 0$. Now $R_1 = 6$ tons and stress $L_0U_1 = +8.48$ tons. To calculate the stresses in L_0L_1 and L_1L_2 take the center of moments at U_1, and pass a section cutting members U_1U_2, U_1L_2 and L_1L_2, and cutting away the truss to the right. Then assume the stress in L_1L_2 as an external force acting from the outside toward the cut section, and $L_1L_2 \times 20 - R_1 \times 20 = 0$. Now $R_1 = 6$ tons and the stress in $L_0L_1 = L_1L_2 = -6$ tons. To calculate the stress in U_1U_2 take the center of moments at L_2, and pass a section cutting members U_1U_2, U_2L_2 and L_2L_2', and cutting away the truss to the right. Then assume the stress in L_1U_2 as an external force acting from the outside toward the cut section, and $U_1U_2 \times 24.25 - R_1 \times 40 + W \times 20 = 0$. Now $R_1 = 6$, $W = 3$ tons, and the stress in $U_1U_2 = +7.42$ tons. To calculate the stress in U_1L_2 take the center of moments at A, and pass a section cutting members U_1U_2, U_1L_2, and L_1L_2, and cutting away the truss to the right. Then assume the stress in U_1L_2 as an

external force acting from the outside toward the cut section, and $U_1L_2 \times 70.7 + R_1 \times 60 - W \times 80 = 0$. Now $R_1 = 6$ tons and $W = 3$ tons, and $U_1L_2 \times 70.7 = -120$ ft.-tons, and stress $U_1L_2 = -1.70$ tons. The other dead load stresses are calculated as shown.

Live Load Stresses.—The live load chord stresses are equal to the dead load chord stresses multiplied by 8/3. The maximum stress in U_1L_2 will occur with loads at L_2, L_2', and L_1', while the maximum stress in counter U_2L_1 will occur with a load at L_1 only. The maximum tension in U_2L_2 will occur with all the live loads on the bridge, while the maximum compression will occur when there is a maximum stress in the counter U_2L_2', loads at L_2' and L_1'. The details of the solution are shown in the problem.

(*c*) **Results.**—The stress in the counter U_2L_2' and the chords U_2U_2' and L_2L_2' may be calculated by the method of coefficients, and will be the same as for a truss with parallel chords having a depth of 25′ 0″. The maximum stress in U_2L_2' will occur with loads L_2' and L_1' on the bridge, when the left reaction equals $2 \times 3P/5 = \frac{6}{5}P$. The stress in $U_2L_2' = -\frac{6}{5}P \cdot \sec\theta = -6.15$ tons.

Problem 8. Maximum and Minimum Stresses in a Through Warren Truss by Graphic Moments.

(*a*) **Problem.**—Given a through Warren truss, span 140′ 0″, panel length 20′ 0″, depth 20′ 0″, dead load 800 lb. per lineal foot per truss, live load 1,200 lb. per lineal foot per truss. Calculate the maximum and minimum stresses by graphic moments. Scale of truss, 1″ = 20′ 0″. Scale of loads, 1″ = 50,000 lb.

(*b*) **Methods.** *Chord Stresses.*—Calculate the center ordinate of the parabola $= w \cdot L^2/8d = 98{,}000$ lb., and lay it off at 5 to the prescribed scale. Now lay off the vertical line 1–5 at the left and right abutments. Make 1–2 = 2–3 = 3–4 = 2 (4–5). Draw the inclined lines 1–5, 2–5, 3–5, 4–5, 5–5. The intersections of these lines with verticals let drop from the lower chord points are points in the stress parabola for the upper chord stresses. The stresses in the lower chords are the arithmetical means of the stresses in the upper chords on each side. By changing the scale the live load stresses may be scaled directly from the diagram.

Web Stresses.—At the distance of a panel to the left of the left abutment lay off the vertical line 1–8 equal to one-half the total live load on the truss, to the prescribed scale, equal 1,200 × 70 = 84,000 lbs. Now divide the line 1–8 into as many equal parts as there are panels in the truss, and mark the points of division 2, 3, 4, etc. Connect these points of division with the panel point 7, the first panel point to the left of the right abutment. Drop verticals from the panel points of the lower chord of the truss to the line 1–8, and the intersections of like numbered lines will give points on the curve of maximum live load shears.

To construct the dead load shear diagram, lay off $3W$, downward to the prescribed scale under the left abutment, and reduce the shear under each load to the right by W, until the dead load shear is $-3W$ at the right abutment. The dead load shear diagram is then constructed as shown.

Maximum and Minimum Web Stresses.—The maximum shear in any panel is then the ordinate to the right of the panel point on the left end of the panel, and the stresses in the web members are calculated by drawing lines parallel to the corresponding member as shown. Negative stresses are measured downwards from the live load shear curve, and positive stresses are measured upwards from the live load shear curve.

(*c*) **Results.**—This method is an excellent one for illustrating the effect of the different systems of loads, but consumes too much time to be of practical use. It should be noted that the maximum ordinate to the chord parabola is not a chord stress in a Warren truss with an odd number of panels.

Problem 9. Maximum and Minimum Stresses in a Petit Truss by Algebraic Moments.

(*a*) **Problem.**—Given a Petit truss, span 350′ 0″, panel length 25′ 0″, depth at the hip 50′ 0″, depth at center 58′ 0″, dead load 0.9 tons per lineal foot per truss, live load 1.4 tons per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic moments. Scale of truss, 1″ = 40′ 0″. Scale of lever arms, any convenient scale.

(*b*) **Methods.**—Construct a truss diagram carefully to scale as shown. Construct one-half the truss to scale on a large piece of paper and calculate the lever arms as shown, and check by scaling from the diagram. The methods of calculation will be shown by two examples:

1. *Stresses in Tie 6–7. Dead Load Stress.*—Pass a section cutting members 7–X, 6–7, and 6–Y, and cutting away the truss to the right. The center of moments will be at A, the intersection of chords 7–X and 6–Y. Now assume the stress in 6–7 as an external force acting from the outside toward the cut section. Then for equilibrium $6\text{–}7 \times 477.0 + R_1 \times 575 - 3W$

$\times$ 625 = 0. Now R_1 = 146.25 tons and W = 22.5 tons, and solving the equation gives stress 6–7 = − 87.8 tons.

Live Load Stresses.—The maximum live load stress in 6–7 will occur with the longer segment of the truss loaded. Taking moments about point A as for the dead loads the maximum live load stress 6–7 $\times$ 477.0 + $R_1 \times$ 575 = 0. Now R_1 = 55/14 $\times$ 35 tons = 137.5 tons, and the stress in 6–7 = − 165.8 tons.

The minimum live load stress in 6–7 will occur with the shorter segment of the truss loaded. Taking moments about the point A, 6–7 $\times$ 477.0 + $R_1 \times$ 575 − 3$P \times$ 625 = 0. Now R_1 = 90 tons, P = 35 tons, and stress in 6–7 = + 29.1 tons.

2. *Stresses* in *Tie 4–7. Dead Load Stress.*—Pass a section cutting members 7–X, 4–7, 4–5 and 5–Y, and cutting away the truss to the right. Now assume the stress in 4–7 as an external force acting from the outside toward the cut section. Then for equilibrium about the point A, stress 4–7 $\times$ 477.0 + $R_1 \times$ 575 − stress 4–5 $\times$ 442.0 − 2$W \times$ 612.5 = 0. Now the member 4–5 will carry one-half the load carried by 5–6, and the stress equals 1/2 $\times$ 22.5 $\times$ 1.414 = + 15.9 tons. R_1 = 146.25 tons, and 2W = 45 tons. Then stress 4–7 = − 103.6 tons.

Live Load Stresses.—The maximum live load stress in 4–7 will occur with the longer segment loaded. Taking moments about A as for dead loads, stress 4–7 $\times$ 477.0 + $R_1 \times$ 575 − stress 4–5 $\times$ 442.0 = 0. Now stress 4–5 = + 24.8 tons, and R_1 = 66/14 $\times$ 35 = 165 tons. Then stress 4–7 = − 175.7 tons.

The minimum live load stress in 4–7 will occur with two loads to the left of the panel. Taking moments about the point A, the stress 4–7 $\times$ 477.0 + $R_1 \times$ 575 − 2$P \times$ 612.5 = 0 Now R_1 = 62.5 tons and 2P = 70 tons. Then stress 4–7 = + 14.5 tons.

The stresses in the members in the first and second panels and in the two middle panels may be calculated by coefficients. Check up the dead load chord stresses by comparing with the stresses obtained by graphic resolution in Problem 2.

(*c*) **Results.**—The auxiliary members carry the stresses directly toward the abutments and there is no ambiguity of loading as in the case of a truss subdivided as in Problem 6. However, the method of subdividing shown in Problem 6 is used in preference to that shown in this problem. The Petit truss is quite generally used for long span pin-connected highway and railway bridges.

PROBLEM 10. LIVE LOAD STRESSES IN A THROUGH PRATT TRUSS FOR COOPER'S E 60 LOADING.

(*a*) **Problem.**—Given a Pratt truss, span 165′ 0″, panel length 23′ 6$\frac{7}{8}$″, depth 30′ 0″, live load Cooper's E 60 loading. Calculate the position of the loads and the maximum and minimum stresses due to the prescribed loading by algebraic moments. Scale of truss, 1″ = 25′ 0″.

(*b*) **Methods.** *Chord Stresses.*—Calculate the position of the wheels for a maximum bending moment at the different joints in the lower chord. The criterion for maximum bending moment at any joint in a Pratt truss is, "the average load on the left of the section must be the same as the average load on the entire bridge." Having determined the wheel that is at the joint for a maximum moment, calculate the maximum bending moment as shown Having calculated the maximum bending moments, the chord stresses are found by dividing the bending moment by the depth of the truss. The moment diagram is given in Table V*b*, Chapter IV.

Web Stresses.—Calculate the position of the wheels for maximum shears in the different panels. The criterion for maximum shear in a panel is, "the load on the panel must equal the load on the bridge divided by the number of panels." The criterion for maximum bending moment at L_1 is the same as the criterion for maximum shear in panel L_0L_1. Having determined the position of the wheels for maximum shears in the different panels, calculate the maximum shears as shown. The stress in a web is equal to the shear in the panel multiplied by sec θ.

Floorbeam Reaction.—The stress in the hip vertical U_1L_1 is equal to the maximum floorbeam reaction. This is calculated as follows: Take a simple beam with a span equal to the sum of two panel lengths and calculate the maximum bending moment at the point in the beam corresponding to the panel point; in this case it will be the center of the span. This bending moment multiplied by the sum of the panel lengths divided by the product of the panel lengths will be the maximum floorbeam reaction; in this case the maximum bending moment at the center will be multiplied by 2 divided by the panel length.

(*c*) **Results.**—When the maximum stresses occur in chords U_2U_3, U_3U_3' and L_3L_3', counter $U_3'L_4$ is in action. It occasionally happens that there is more than one position of the loading that will satisfy the criterion for maximum bending moment. In this case the moments for each loading must be calculated.

PROBLEM 11. STRESSES IN THE PORTAL OF A BRIDGE BY ALGEBRAIC MOMENTS AND GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given the portal of a bridge of the type shown, inclined height 30′ 0″, center to center width 15′ 0″, load R = 2,000 lb., end-posts pin-connected at the base. Calculate the stresses by algebraic moments and check by graphic resolution. Scales as shown.

(*b*) **Methods.**—Now $H = H' = 1,000$ lb. $V = -V'$, and by taking moments about B, $V = 30 \times 2,000/15 = 4,000$ lb. $= -V'$.

Algebraic Moments.—In passing sections care should be used to avoid cutting the end-posts for the reason that these members are subject to bending stresses in addition to the direct stresses. To calculate the stress in member 3–Y take the center of moments at joint (1) and pass a section cutting members 4–b, 3–4 and 3–Y, and cutting the portal away to the left of the section. Then assume stress 3–Y as an external force acting from the outside toward the cut section, and $3\text{-}Y \times 10 \times 0.447 + H \times 30' = 0$. The stress in 3–$Y$ = – 6,710 lb. The remaining stresses are calculated as shown.

Graphic Resolution.—Lay off $a\text{-}A = A\text{-}b = H = 1,000$ lb., and $A\text{-}Y = V'' = 4,000$ lb. Then beginning at point B in the portal the force polygon for equilibrium is $a\text{-}A\text{-}Y\text{-}1'\text{-}a$, in which $1'\text{-}a$ is the stress in the auxiliary member 1–a, and $Y\text{-}1'$ is the stress in the post 1–Y when the auxiliary member is acting. The true stress in 1–Y is equal to the algebraic sum of the vertical components of the stress $1'\text{-}a$ and $Y\text{-}1'$, and equals $V' = -4,000$ lb. Next complete the force triangle at the intersection of the auxiliary members. Stress $1'\text{-}a$ is known and the force triangle is $a\text{-}1'\text{-}2'\text{-}a$, the forces acting as shown. The stress diagram is carried through in the order shown, checking up at the point A. The correct stresses are shown by the full lines in the stress diagram. The true stress in 3–2 will produce equilibrium for vertical stresses at joint (1) as shown. The maximum shear in the posts is $H = 1,000$ lb. The maximum bending moment in the posts will occur at the foot of the member 3–Y, joint (3), and is $M = 1,000 \times 20 \times 12 = 240,000$ in.-lb

(*c*) **Results.**—The method of graphic resolution requires less work and is more simple than the method of algebraic moments.

Note: The portal is not pin-connected at joints (3) and the corresponding joint on the opposite side, as might be inferred from the figure.

PROBLEM 12. WIND LOAD STRESSES IN A TRESTLE BENT.

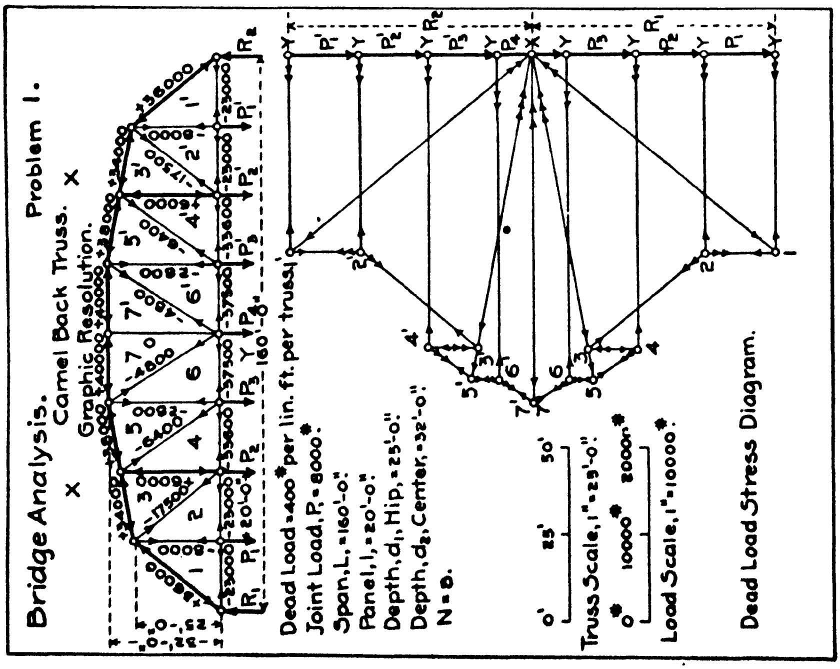
(*a*) **Problem.**—Given a trestle bent, height 45′ 0″, width at the base 30′ 0″, width at the top 9′ 0″, wind loads P_0, P_1, P_2, P_3, P_4, as shown. Calculate the stresses in the members of the bent due to wind loads by algebraic moments, and check by calculating the stresses by graphic resolution. Assume that the diagonal members are tension members, and that the dotted members are not acting for the wind blowing as shown. Scale of truss, 1″ = 10′ 0″. Scale of loads, 1″ = 2,000 lb.

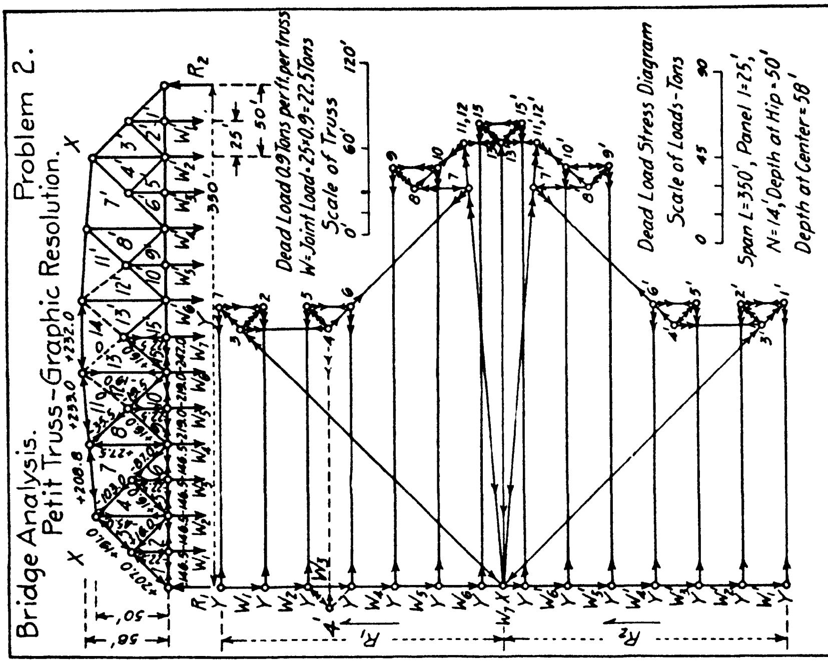
(*b*) **Methods.**—*Algebraic Moments.*—To calculate the stresses in the diagonal members take centers of moments about the point A, the point of intersection of the inclined posts. Then to calculate the stress in 3–4, pass a section cutting members 3–X, 3–4 and 4–Y; assume that the stress in 3–4 is an external force acting from the outside toward the cut section, and $3\text{-}4 \times 15.9' + 3,000 \times 19.3' + 3,000 \times 11.3' = 0$. The stress 3–4 = – 5,800 lb. Stresses in 4–5, 5–6, 6–7, 7–8 and 8–Z are calculated in a similar manner. To obtain reaction R_1 take moments about R_2, and $R_1 \times 30' - 2,000 \times 15' - 2,000 \times 30' - 3,000 \times 45' - 3,000 \times 53' = 0$. Then $R_1 = 12,800$ lb. $= -R_2$.

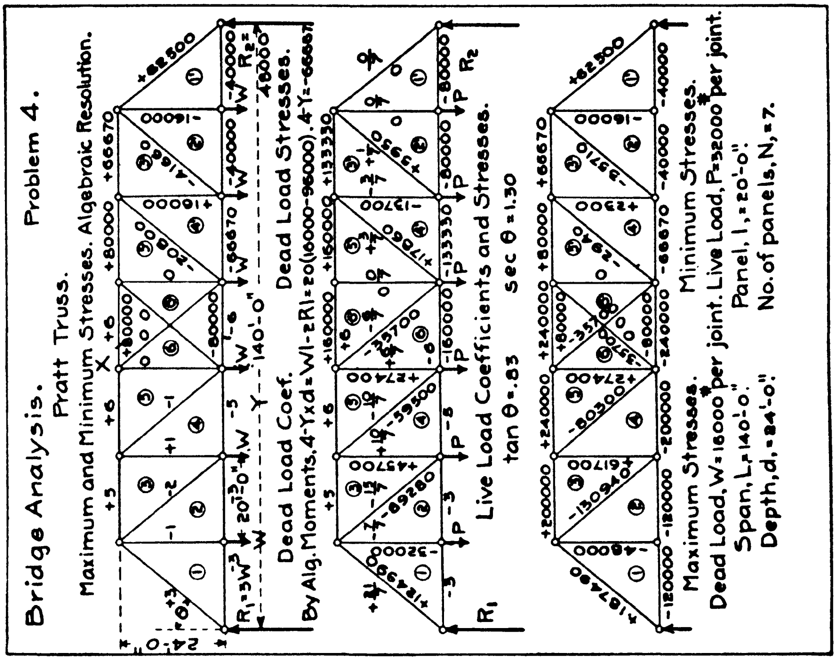
To calculate the stress in 4–Y, take center of moments at joint P_2, and pass a section cutting members 5–X, 4–5 and 4–Y, and assume the stress in 4–Y as an external force acting from the outside toward the cut section. Then $4\text{-}Y \times 15.6' - 3,000 \times 15' - 3,000 \times 23' = 0$. Then 4–$Y$ = + 7,300 lb.

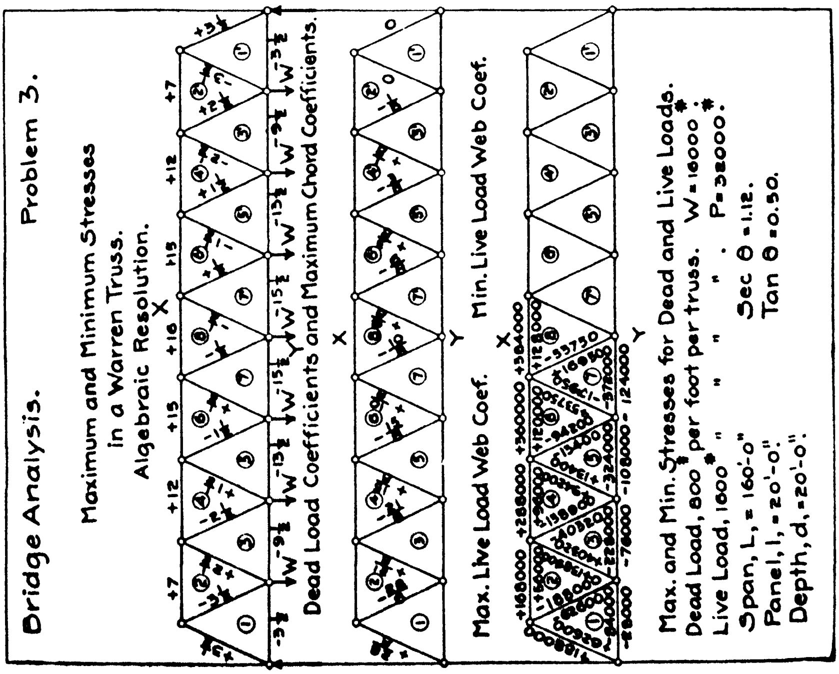
Graphic Resolution.—The load P_0 is assumed as transferred to the bent by means of the auxiliary members. The loads P_0, P_1, P_2, P_3, P_4 are laid off as shown, and with the load P_0 the stress triangle $Y\text{-}X\text{-}2$ is drawn. The remainder of the solution is easily followed.

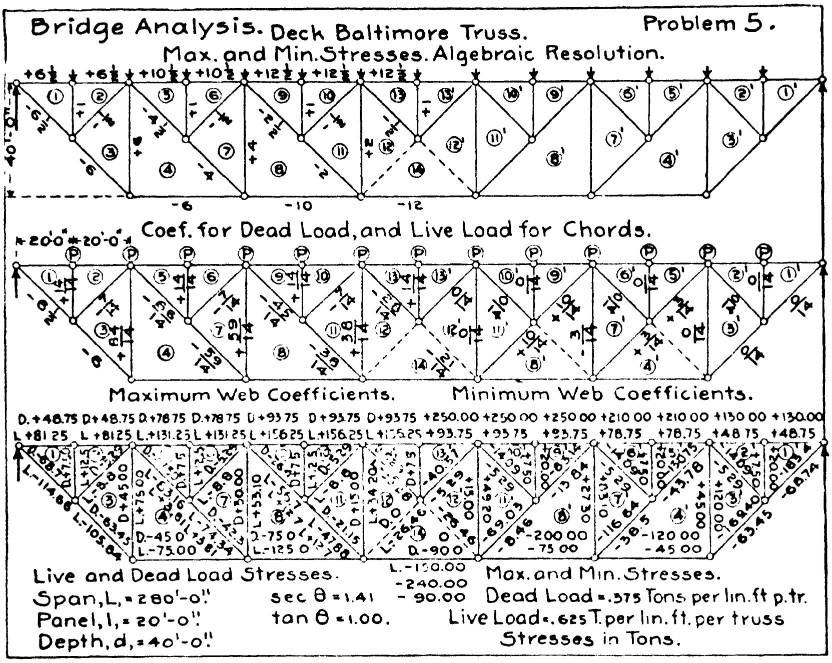
(*c*) **Results.**—The stress in the auxiliary member 2–Y acts as a load at the top of post 4–Y. Load P_0 is the wind load on the train and is transferred to the rails by the car. For the reason that the wind may blow from the opposite direction, both sets of stresses must be considered in combination with the dead and live load stresses in designing the columns.

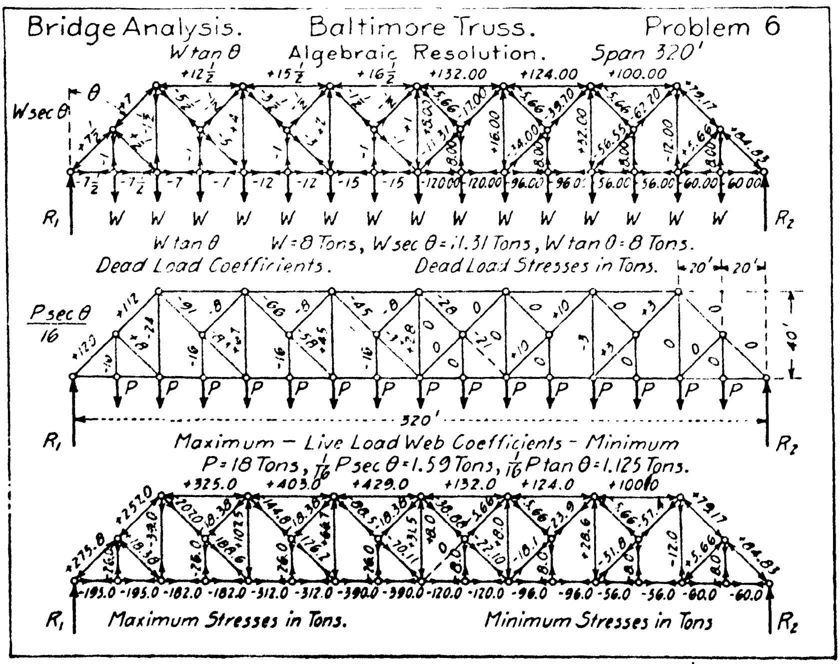

Bridge Analysis.
Problem 1.
Camel Back Truss.
Graphic Resolution.
Dead Load = 400# per lin. ft. per truss.
Joint Load, P = 8000#.
Span, L, = 160'-0".
Panel, l, = 20'-0".
Depth, d1, Hip, = 25'-0".
Depth, d2, Center, = 32'-0".
N = 8.
Truss Scale, 1" = 25'-0".
Load Scale, 1" = 10000#.
Dead Load Stress Diagram.

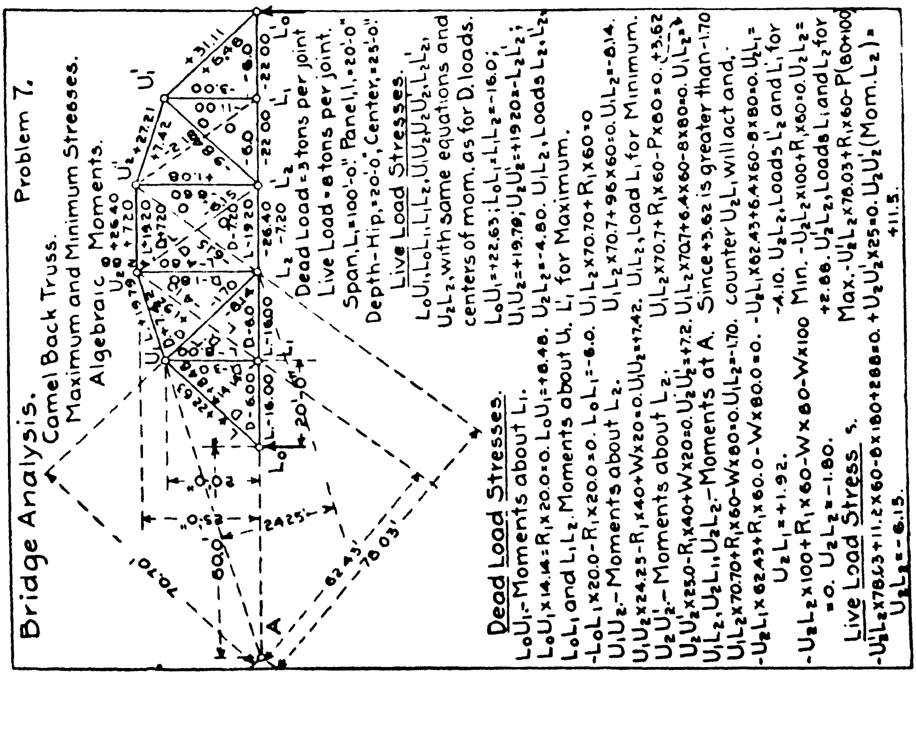

Bridge Analysis.
Petit Truss-Graphic Resolution.
Problem 2.
Dead Load 0.9 Tons per ft. per truss
W = Joint Load = 25×0.9 = 22.5 Tons
Scale of Truss
Dead Load Stress Diagram
Scale of Loads-Tons
Span L=350', Panel l=25',
N=14, Depth at Hip=50'
Depth at Center=58'

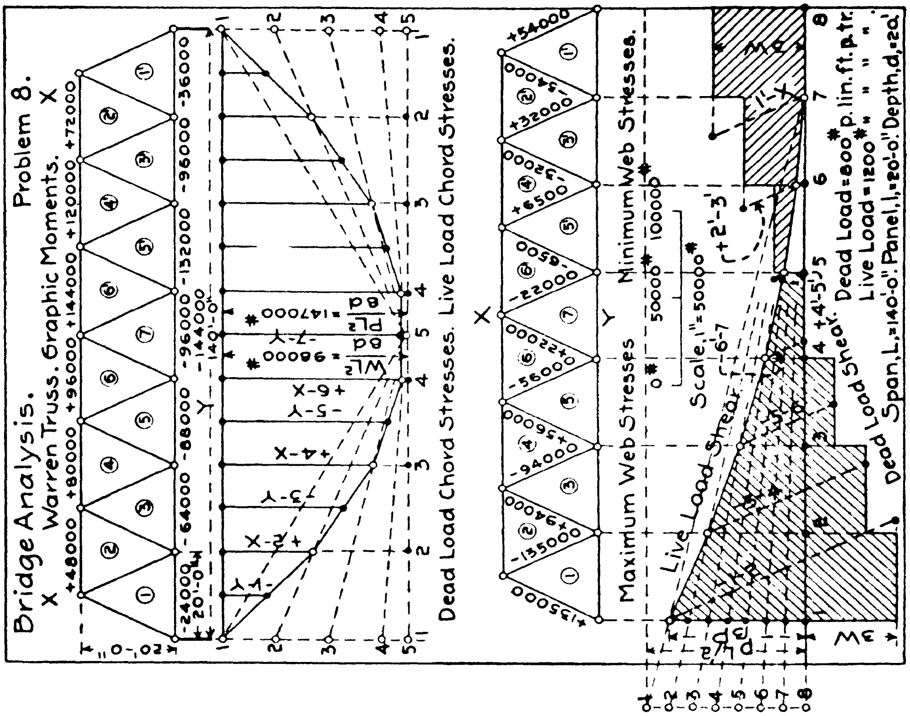

Bridge Analysis.
Problem 4.
Pratt Truss.
Maximum and Minimum Stresses. Algebraic Resolution.
Dead Load Coef.
Dead Load Stresses.
Live Load Coefficients and Stresses.
tan θ = .83
sec θ = 1.30
Maximum Stresses.
Minimum Stresses.
Dead Load, W = 16000# per joint. Live Load, P = 32000# per joint.
Span, L, = 140'-0".
Panel, l, = 20'-0".
Depth, d, = 24'-0".
No. of panels, N, = 7.

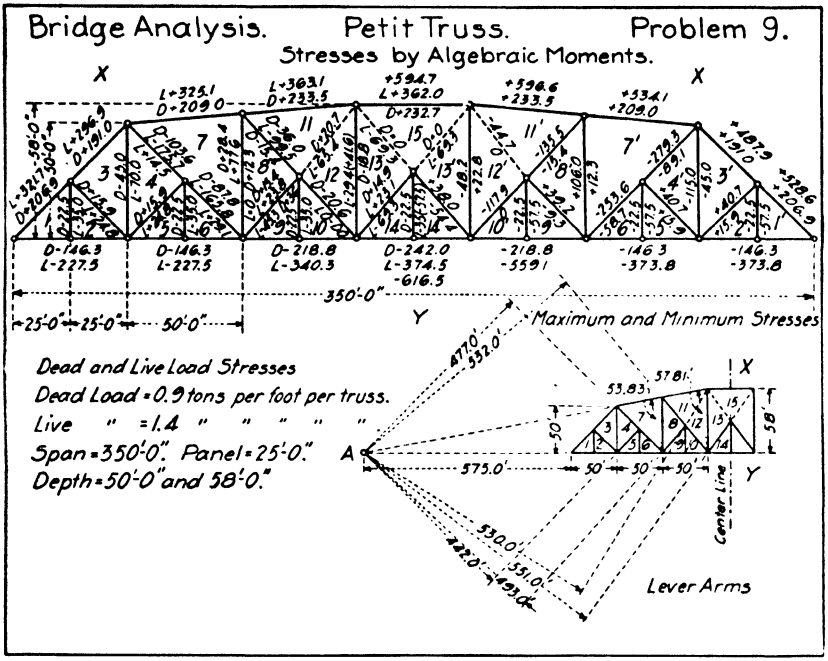

Bridge Analysis.
Problem 3.
Maximum and Minimum Stresses
in a Warren Truss.
Algebraic Resolution.
Dead Load Coefficients and Maximum Chord Coefficients.
Max. Live Load Web Coef.
Min. Live Load Web Coef.
Max. and Min. Stresses for Dead and Live Loads.
Dead Load, 800# per foot per truss. W = 16000#.
P = 32000#.
Span, L, = 160'-0".
Sec θ = 1.12.
Panel, l, = 20'-0".
Tan θ = 0.50.
Depth, d, = 20'-0".

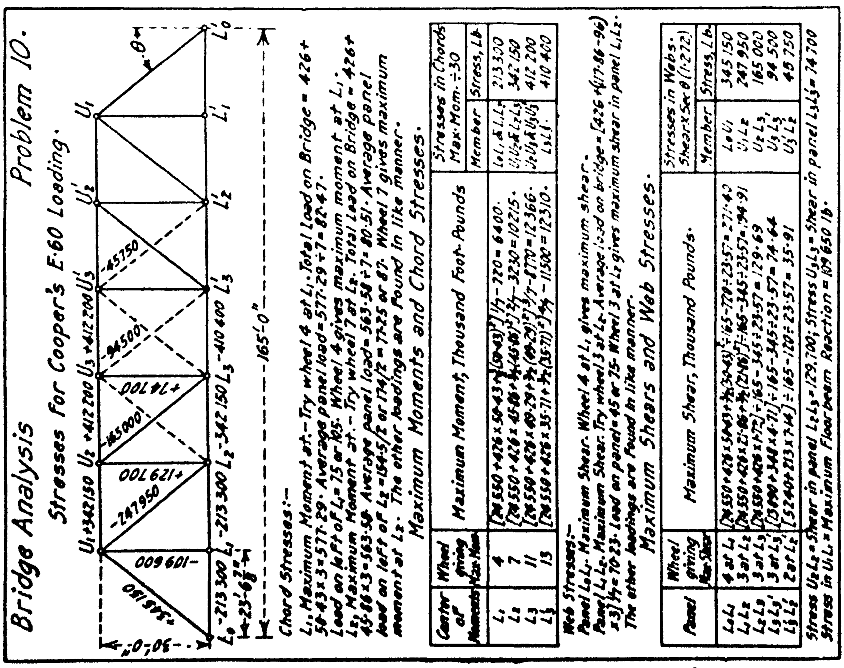

Bridge Analysis. Deck Baltimore Truss. Problem 5.
Max. and Min. Stresses. Algebraic Resolution.
Coef. for Dead Load, and Live Load for Chords.
Maximum Web Coefficients. Minimum Web Coefficients.
Live and Dead Load Stresses.
Span, L, = 280'-0".
Panel, l, = 20'-0".
Depth, d, = 40'-0".
sec θ = 1.41
tan θ = 1.00.
Max. and Min. Stresses.
Dead Load = .375 Tons per lin. ft p. tr.
Live Load = .625 T. per lin. ft. per truss
Stresses in Tons.

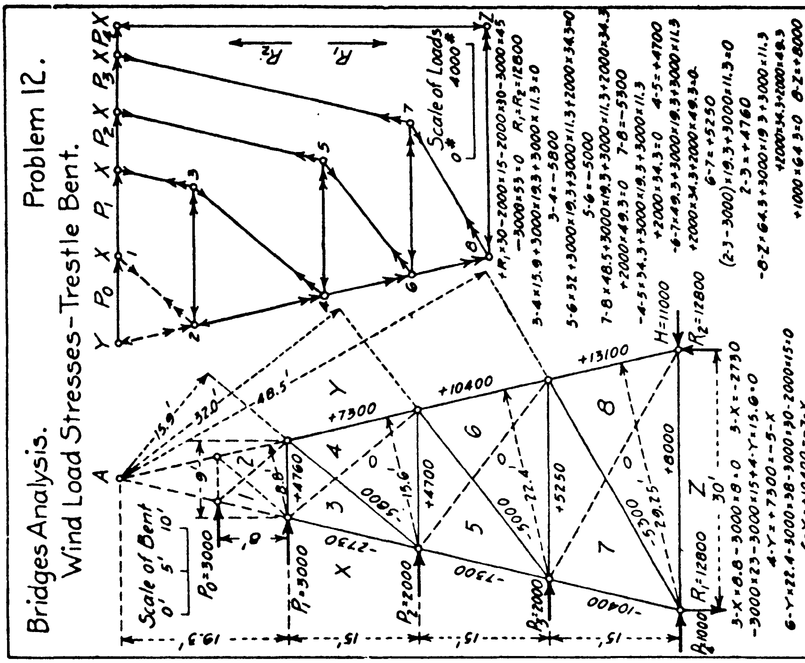

Bridge Analysis. Baltimore Truss. Problem 6
W tan θ Algebraic Resolution. Span 320'
W sec θ
W tan θ W = 8 Tons, W sec θ = 11.31 Tons, W tan θ = 8 Tons.
Dead Load Coefficients. Dead Load Stresses in Tons.
P sec θ / 16
320'
Maximum — Live Load Web Coefficients — Minimum
P = 18 Tons, 1/16 P sec θ = 1.59 Tons, 1/16 P tan θ = 1.125 Tons.
Maximum Stresses in Tons. Minimum Stresses in Tons

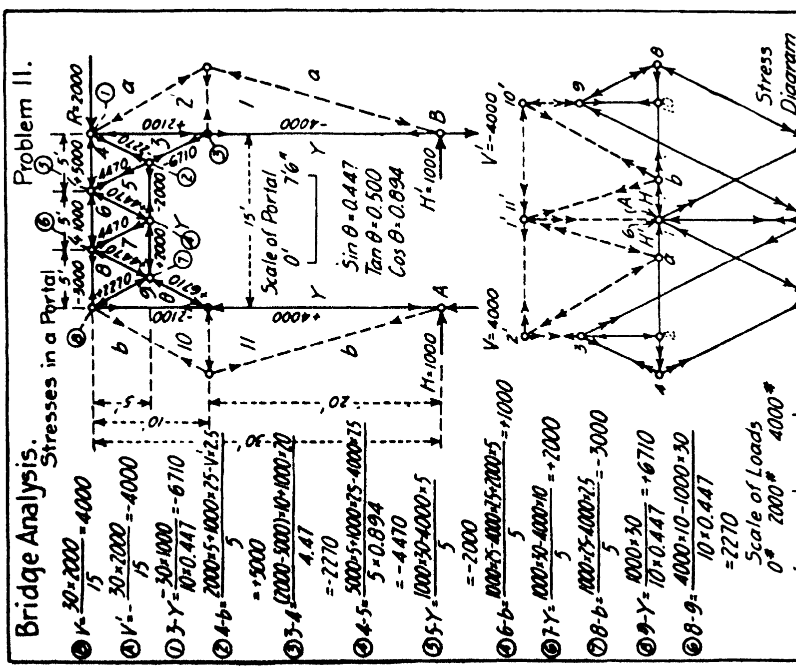

Bridge Analysis. Problem 7.
Camel Back Truss.
Maximum and Minimum Stresses.
Algebraic Moments.
Dead Load = 3 tons per joint.
Live Load = 8 tons per joint.
Span, L, = 100'-0". Panel, l, = 20'-0"
Depth-Hip, = 20'-0", Center, = 25'-0".
Live Load Stresses.
L₀U₁, L₀L₁, L₁L₂, U₁U₂, U₂U'₂, L₂L'₂, U₂L₂, with same equations and centers of mom. as for D. loads.
L₀U₁ = +22.63; L₀L₁ = L₁L₂ = -16.0; U₁U₂ = +19.79; U₂U'₂ = +19.20 = -L₂L'₂; U₂L₂ = -4.80. U₁L₂, Loads L₂, L'₂, L'₁ for Maximum.
U₁L₂x70.70+R₁x60=0
U₁L₂x70.7+96x60=0. U₁L₂=-8.14.
U₁L₂, Load L₁ for Minimum.
U₁L₂x70.7+R₁x60-Px80=0. +3.62
U₁L₂x70.7+6.4x60-8x80=0. U₁L₂=
Since +3.62 is greater than -1.70 counter U₂L₁ will act and,
-U₂L₁x62.43+6.4x60-8x80=0. U₂L₁= -4.10. U₂L₂, Loads L'₂ and L'₁ for Min. -U₂L₂x100+R₁x60=0. U₂L₂= +2.88. U'₂L₂, Loads L₁ and L₂ for Max. -U'₂L₂x78.03+R₁x60-P(80+100) +U₂U'₂x25=0. U₂U'₂(Mom. L₂) = +11.5.
Dead Load Stresses.
L₀U₁.-Moments about L₁.
L₀U₁x14.14=R₁x20.0=0. L₀U₁=+8.48.
L₀L₁ and L₁L₂. Moments about U₁.
-L₀L₁x20.0-R₁x20.0=0. L₀L₁=-6.0.
U₁U₂.-Moments about L₂.
U₁U₂x24.25-R₁x40+Wx20=0. U₁U₂=+7.42.
U₂U'₂.-Moments about L₂.
U₂U'₂x25.0-R₁x40+Wx20=0. U₂U'₂=+7.2.
U₁L₂, U₂L₁, U₂L₂.-Moments at A.
U₁L₂x70.70+R₁x60-Wx80=0. U₁L₂=-1.70.
-U₂L₁x62.43+R₁x60.0-Wx80.0=0.
U₂L₁=+1.92.
-U₂L₂x100+R₁x60-Wx80-Wx100=0. U₂L₂=-1.80.
Live Load Stress s.
-U'₂L₂x78.03+11.2x60-8x180+288=0.
U₂L₂=-6.15.


Bridge Analysis. Problem 8.
X Warren Truss. Graphic Moments. X
+48000 +80000 +96000 +144000 +120000 +72000
-24000 -64000 -88000 -96000 -132000 -96000 -36000
-144000
20'-0"
140'-0"
Dead Load Chord Stresses. Live Load Chord Stresses.
WL²/8d = 98000#
PL²/8d = 147000#
Maximum Web Stresses
Minimum Web Stresses.
Scale, 1" = 50000#
Live Load Shear
Dead Load Shear
Dead Load = 800# p. lin. ft. p. tr.
Live Load = 1200# " " "
Span, L, = 140'-0". Panel, l, = 20'-0". Depth, d, = 20'.


Bridge Analysis.
Petit Truss.
Problem 9.
Stresses by Algebraic Moments.
Dead and Live Load Stresses
Dead Load = 0.9 tons per foot per truss.
Live " = 1.4 " " " " "
Span = 350'-0". Panel = 25'-0".
Depth = 50'-0" and 58'-0".
Maximum and Minimum Stresses
Lever Arms
Center Line


Bridge Analysis
Problem 10.
Stresses for Cooper's E-60 Loading.
Chord Stresses:-
Maximum Moments and Chord Stresses.
Web Stresses:-
Maximum Shears and Web Stresses.


Bridges Analysis.
Wind Load Stresses-Trestle Bent.
Problem 12.
Scale of Bent
Scale of Loads


Bridge Analysis.
Stresses in a Portal.
Problem 11.
Scale of Portal
Sin θ = 0.447
Tan θ = 0.500
Cos θ = 0.894
Stress Diagram
Scale of Loads

STRESSES IN STIFF FRAMES.—Stiff frames with rigid joints are statically indeterminate. In calculating the stresses in frames with stiff joints it is assumed (1) that the members have large sections and that the distortions due to direct stresses are small and may be neglected; (2) that the joints are perfectly rigid; and (3) that deformations due to shear in the members are zero.

If any member of a stiff frame be assumed as cut, for equilibrium we will have from mechanics the following relations existing between the horizontal distance between the cut ends, the vertical distance between the cut ends and the tangents at the cut ends.

The horizontal distance between the cut ends due to any given loading will be

$$x_1 = \sum \frac{M \cdot y}{E \cdot I} ds = 0 \tag{5}$$

The vertical distance between the cut ends will be

$$y_1 = \sum \frac{M \cdot x}{E \cdot I} ds = 0 \tag{6}$$

The angle between the tangents at the cut ends will be

$$\phi = \sum \frac{M}{E \cdot I} ds = 0 \tag{7}$$

Equations (5), (6) and (7) are equated to zero since the cut ends must stay in contact and the member must be continuous.

Equations (5), (6) and (7) may be solved (1) by the Work Method, (calculus method), (2) by the Area Moment Method, or (3) by the Slope Deflection Method. The Area Moment and the Slope Deflection Methods may be used only with frames with straight members. The Work Method may be used for frames with straight and curved members. The Work and Area Moment Methods will be illustrated by means of a simple problem.

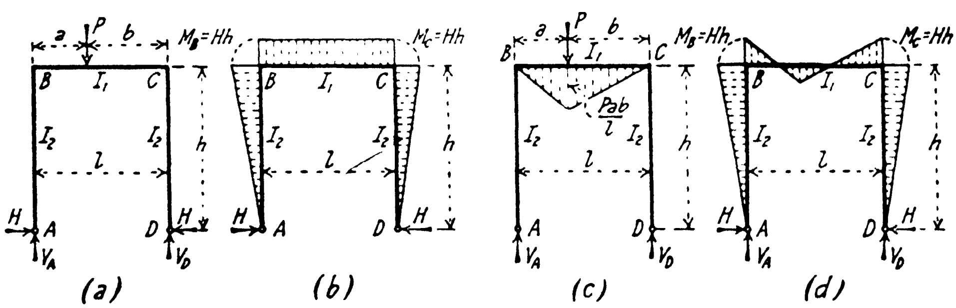


FIG. 4.

Work Method.—The stresses in the portal frame with pin connected columns in Fig. 4 may be calculated by the Work Method as follows:

If the frame is free to move horizontally at D, the movement due to the load P will be

$$\frac{dW}{dH} = \Delta = \int \frac{M \cdot m \cdot ds}{E \cdot I} \tag{8}$$

Now apply one lb. horizontally at D, and the distance that the frame will be brought back by one pound will be

$$\delta' = \int_A^D \frac{m \cdot m \cdot ds}{E \cdot I} \tag{9}$$

A horizontal thrust H will bring the point D back to its original position and

$$\Delta = H\int_A^D \frac{m^2\cdot ds}{E\cdot I} \tag{10}$$

Equating equations (8) and (10), and solving

$$H = \frac{\displaystyle\int_A^D \frac{M\cdot m\cdot ds}{E\cdot I}}{\displaystyle\int_A^D \frac{m^2\cdot ds}{E\cdot I}} \tag{11}$$

From (c), Fig. 4,

$$\int_A^D \frac{M\cdot m\cdot ds}{E\cdot I} = \frac{P\cdot a\cdot b\cdot h}{2E\cdot I_1} \tag{12}$$

From (b), Fig. 4,

$$\int_A^D \frac{m^2\cdot ds}{E\cdot I} = \frac{2h^3}{3E\cdot I_2} + \frac{h^2\cdot l}{E\cdot I_1} \tag{13}$$

and substituting (12) and (13) in (11),

$$H = \frac{3P\cdot a\cdot b}{2h\cdot l\left(2\dfrac{I_1}{I_2} + 3\right)} \tag{14}$$

$$= \frac{3P\cdot a\cdot b}{2h\cdot l(2k + 3)} \tag{15}$$

where $k = I_1/I_2$.

Area Moment Method.—The moment of the negative moment area about the foot of column A will be equal to the moment of the positive moment area about A, and

$$\frac{2H\cdot h \times \tfrac{1}{2}h \times \tfrac{2}{3}h}{I_2} + \frac{H\cdot h^2\cdot l}{I_1} = \frac{P\cdot a\cdot b/l \times \tfrac{1}{2}l \times h}{I_1} \tag{11}$$

and

$$H = \frac{3P\cdot a\cdot b}{2h\cdot l(2k + 3)} \tag{15}$$

For the calculation of the stresses in stiff frames by the Work Method, the Area Moment Method, and the Slope Deflection Method, see the author's "Design of Steel Mill Buildings," Fourth Edition.

Stresses in Stiff Frames.—The moments and stresses in several types of stiff frames are given in Fig. 5 to Fig. 16.

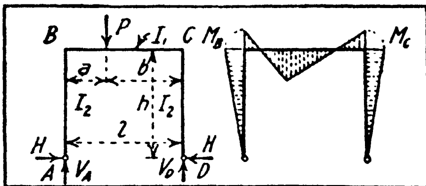
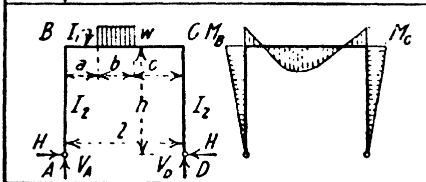
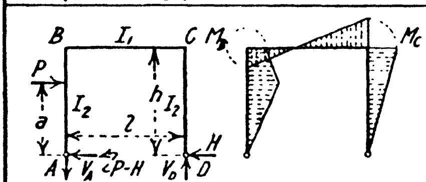
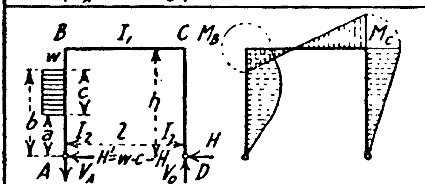
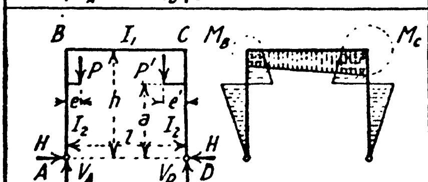
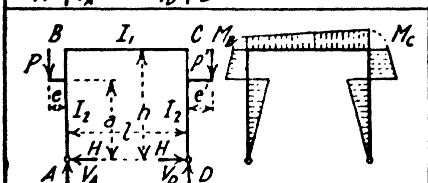
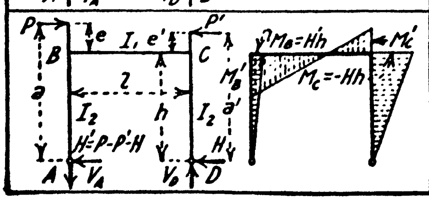
	GENERAL CASE $H=\frac{3Pab}{2hl(2k+3)}$; $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $V_A=\frac{Pb}{l}$; $V_D=\frac{Pa}{l}$ $M_B=M_C=-Hh$	LOAD IN CENTER; $a=b$ $H=\frac{3Pl}{8h(2k+3)}$; $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $V_A=\frac{P}{2}$ $V_D=\frac{P}{2}$. $M_B=M_C=-Hh$
	GENERAL CASE $H=\frac{wb}{4}\cdot\frac{[6ac+b(3l-2b)]}{hl(2k+3)}$; $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $V_A=\frac{wb}{2l}(2c+b)$; $V_D=\frac{wb}{2l}(2a+b)$ $M_B=M_C=-Hh$	TOP FULLY LOADED; $b=l$ $H=\frac{wl^2}{4h(2k+3)}$; $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $V_A=V_D=\frac{wl}{2}$ $M_B=M_C=-Hh$
	GENERAL CASE $H=\frac{Pa}{2}\cdot\frac{3h^2+k(3h^2-a^2)}{h^3(2k+3)}$; $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $V_A=V_D=\frac{Pa}{l}$ $M_B=Pa-Hh$; $M_C=-Hh$	LOAD AT B; $a=h$ $H=\frac{P}{2}$ $V_A=V_D=\frac{Ph}{l}$ $M_B=+\frac{1}{2}Ph$; $M_C=-\frac{1}{2}Ph$
	GENERAL CASE $H=\frac{w[6h^2(1+k)(b^2-a^2)-k(b^4-a^4)]}{8h^3(2k+3)}$ $V_A=V_D=\frac{w(b^2-a^2)}{2l}$ $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $M_B=V_D l-Hh$; $M_C=-Hh$	SIDE FULLY LOADED; $c=h$ $H=\frac{wh}{8}\cdot\frac{6+5k}{2k+3}$ $V_A=V_D=\frac{wh^2}{2l}$, $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $M_B=V_D l-Hh$; $M_C=-hh$
	GENERAL CASE $H=3(Pe+P'e')\frac{k(h^2-a^2)+h^2}{2h^3(2k+3)}$; $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $V_A=\frac{P'e'+P(l-e)}{l}$; $V_D=\frac{Pe+P'(l-e')}{l}$ $M_B=Pe-Hh$; $M_C=P'e'-Hh$	LOAD ON ONE SIDE; $P'=0$ $H=3Pe\frac{k(h^2-a^2)+h^2}{2h^3(2k+3)}$ $V_A=\frac{P(l-e)}{l}$; $V_D=\frac{Pe}{l}$; $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $M_B=Pe-Hh$; $M_C=-Hh$
	GENERAL CASE $H=3(Pe+P'e')\frac{k(h^2-a^2)+h^2}{2h^3(2k+3)}$; $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $V_A=\frac{P(l+e)-P'e'}{l}$; $V_D=\frac{P'(l+e')-Pe}{l}$ $M_B=Hh-Pe$; $M_C=Hh-P'e'$	LOADS AT TOP; $a=h$ $H=\frac{3(Pe+P'e')}{2h(2k+3)}$; $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ V_A and V_D are the same as in general case. For sides, $M_B=M_C=Hh$
	GENERAL CASE $H=\frac{(P-P')2kh+3(Pa+P'a')-6P'h}{2h(2k+3)}$ $V_A=V_D=\frac{Pa-P'a'}{l}$; $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $M_B=H'h$; $M_B'=M_B+Pe$; $M_C'=M_C+P'e'$.	LOAD ON ONE SIDE; $P'=0$. $H=\frac{P(2kh+3a)}{2h(2k+3)}$ $V_A=V_D=\frac{Pa}{l}$; $k=\frac{I_1}{I_2}\cdot\frac{h}{l}$ $M_B'=M_B+Pe$; $M_C'=M_C$

FIG. 5. STRESSES IN STIFF FRAMES.

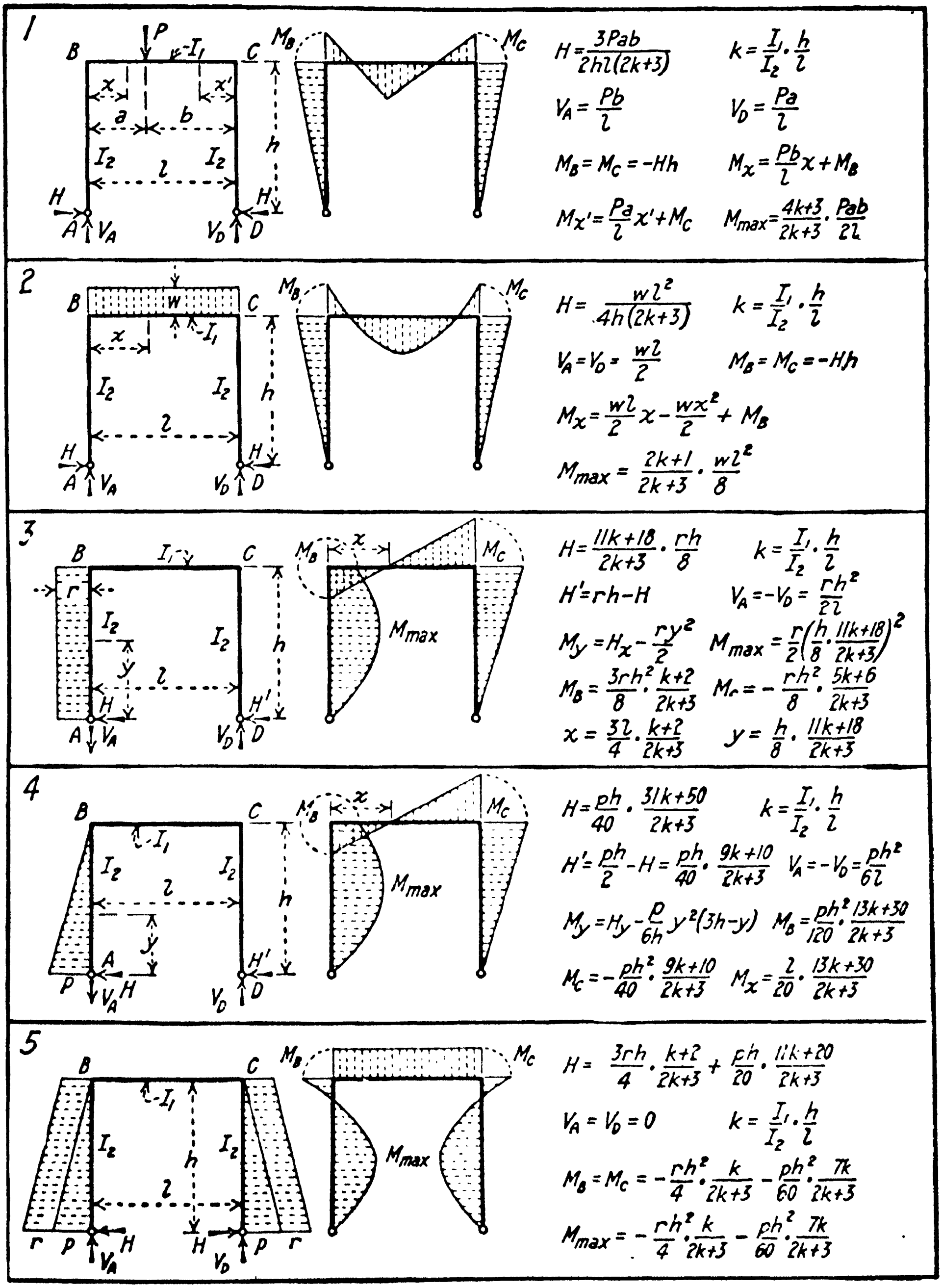
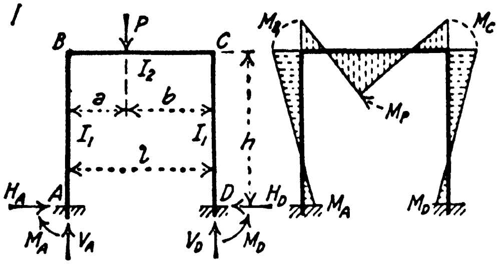


FIG. 6. STRESSES IN STIFF FRAMES.

1



$$V_A=\frac{Pb}{l}\cdot\frac{1+s-2s^2+6k}{1+6k}\qquad k=\frac{I_2}{I_1}\cdot\frac{h}{l}$$

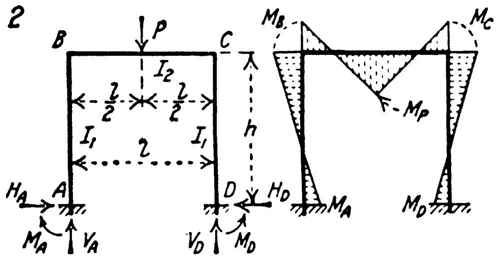
$$V_D=\frac{Pa}{l}\cdot\frac{3s-2s^2+6k}{1+6k}\qquad s=\frac{a}{l}$$

$$H_A=H_D=\frac{3Pab}{2hl(2+k)}\qquad M_A=\frac{Pab}{2l}\cdot\frac{5k-1+2s(2+k)}{(2+k)(1+6k)}$$

$$M_D=\frac{Pab}{2l}\cdot\frac{3+7k-2s(2+k)}{(2+k)(1+6k)}\qquad M_B=-M_A+H_Ah$$

$$M_C=-M_D+H_Dh\qquad M_P=-M_A+H_Ah-V_Aa$$

2



$$V_A=V_D=\frac{F}{2}\qquad k=\frac{I_2}{I_1}\cdot\frac{h}{l}$$

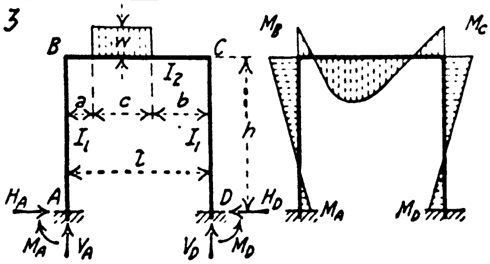
$$H_A=H_D=\frac{3Pl}{8h(2+k)}$$

$$M_A=M_D=\frac{Pl}{8(2+k)}=H_A\cdot\frac{h}{3}$$

$$M_B=M_C=-\frac{Pl}{4(2+k)}=-H_A\cdot\frac{2h}{3}$$

$$M_P=\frac{Pl}{4}+M_B=\frac{Pl}{4}\cdot\frac{1+k}{2+k}$$

3



$$V_A=\frac{wc}{l^3(1+6k)}\Big[3abc+b^2(3l-2b)+c^2\Big(l+b-\frac{c}{2}\Big)+3l^2(2b+c)k\Big]$$

$$V_D=wc-V_A\qquad k=\frac{I_2}{I_1}\cdot\frac{h}{l}$$

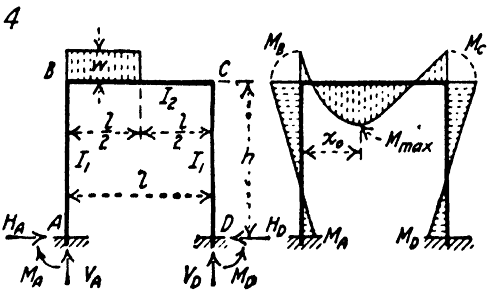
$$H_A=H_D=\frac{wc}{4hl(2+k)}(6ab+3cl-2c^2)$$

$$M_A=\frac{w}{4l^2(2+k)(1+6k)}\Big[6abcl+4a^2c^2+8ab^2c+\frac{10}{3}\cdot c^3l-c^2l^2-2c^4+(14abcl+2a^2c^2-4ab^2c-2c^3l+5c^2l^2-c^4)k\Big]$$

$$M_D=\frac{w}{4l^2(2+k)(1+6k)}\Big[6abcl+4b^2c^2+8a^2bc+\frac{10}{3}\cdot c^3l-c^2l^2-2c^4+(14abcl+2b^2c^2-4a^2bc-2c^3l+5c^2l^2-c^4)k\Big]$$

$$M_B=-M_A-H_Ah\qquad M_C=-M_D-H_Dh$$

4



$$V_A=\frac{wl}{32}\cdot\frac{13+72k}{1+6k}=\frac{wl}{2}-V_D$$

$$V_D=\frac{3wl}{32}\cdot\frac{1+8k}{1+6k}\qquad k=\frac{I_2}{I_1}\cdot\frac{h}{l}$$

$$H_A=H_D=\frac{wl^2}{8h(2+k)}$$

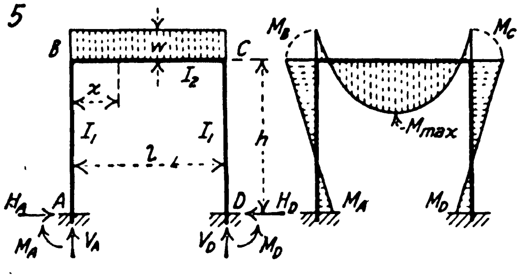
$$M_A=\frac{wl^2}{192}\cdot\frac{2+45k}{(2+k)(1+6k)}\qquad M_D=\frac{wl^2}{192}\cdot\frac{14+51k}{(2+k)(1+6k)}$$

$$M_B=-M_A-H_Ah\qquad M_C=-M_D-H_Dh$$

$$x_0=\frac{13+72k}{1+6k}\cdot\frac{l}{32}=\frac{V_A}{w}$$

FIG. 7. STRESSES IN STIFF FRAMES WITH FIXED COLUMNS.

5



$$V_A = V_D = \frac{wl}{2} \qquad k = \frac{I_2}{I_1} \cdot \frac{h}{l}$$

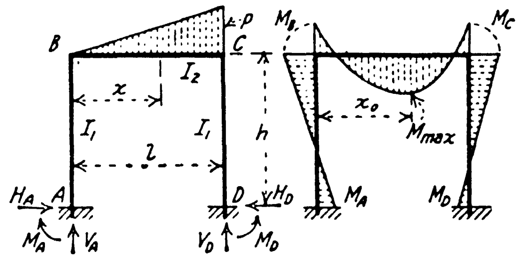
$$H_A = H_D = \frac{wl^2}{4h(2+k)}$$

$$M_A = M_D = \frac{+wl^2}{12(2+k)} \qquad M_B = M_C = \frac{-wl^2}{6(2+k)}$$

$$M_x = \frac{wl}{2} x - \frac{wx^2}{2} - \frac{wl^2}{6(2+k)}$$

$$M_{max} = \frac{wl^2}{8} - \frac{wl^2}{6(2+k)} = \frac{wl^2}{24} \cdot \frac{2+3k}{2+k}$$

6



$$V_A = \frac{pl}{20} \cdot \frac{3+20k}{1+6k} \qquad V_D = \frac{pl}{20} \cdot \frac{7+40k}{1+6k}$$

$$H_A = H_D = \frac{pl^2}{8h(2+k)} \qquad k = \frac{I_2}{I_1} \cdot \frac{h}{l}$$

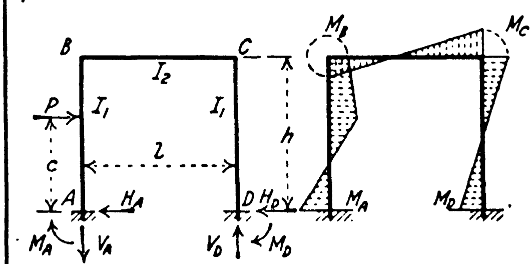
$$M_A = \frac{pl^2}{120} \cdot \frac{7+31k}{(2+k)(1+6k)} \qquad M_D = \frac{pl^2}{120} \cdot \frac{3+29k}{(2+k)(1+6k)}$$

$$M_B = -M_A - H_A h \qquad M_C = -M_D - H_D h$$

$$M_x = M_B + V_A x - \frac{px^3}{6l}$$

$$x_0 = l\sqrt{\frac{0.3+2k}{1+6k}}$$

7



$$-V_A = V_D = \frac{3Pcks}{l(1+6k)} \qquad k = \frac{I_2}{I_1} \cdot \frac{h}{l} \qquad s = \frac{c}{h}$$

$$H_A = P - H_D \qquad H_D = \frac{Ps^2}{2(2+k)}\left[3(1+k) - s(1+2k)\right]$$

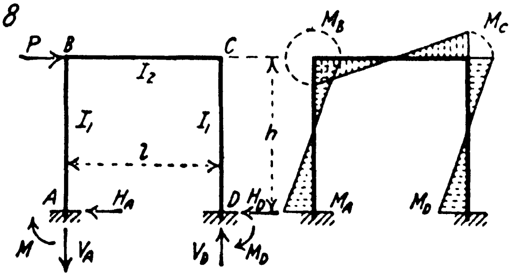
$$M_A = \frac{-Pcs}{2}\left[\frac{2}{s} - \frac{3+2k-s(1+k)}{2+k} - \frac{3k}{1+6k}\right]$$

$$M_D = -\frac{Pcs}{2}\left[\frac{3+2k-s(1+k)}{2+k} - \frac{3k}{1+6k}\right]$$

$$M_B = -M_A - H_D h + Pc$$

$$M_C = -M_D - H_D h \qquad -M_P = M_A + H_A c$$

8



$$-V_A = V_D = \frac{3Phk}{l(1+6k)} \qquad k = \frac{I_2}{I_1} \cdot \frac{h}{l}$$

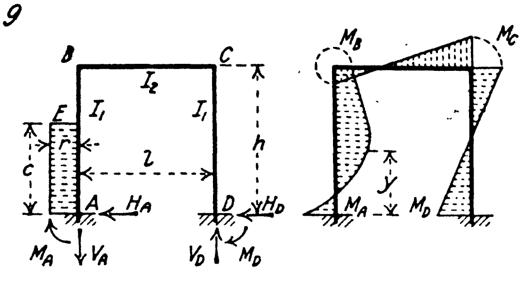
$$H_A = H_D = \frac{P}{2}$$

$$M_A = \frac{-Ph}{2} \cdot \frac{1+3k}{1+6k} \qquad M_D = -\frac{Ph}{2} \cdot \frac{1+3k}{1+6k}$$

$$M_B = -\frac{Ph}{2} \cdot \frac{3k}{1+6k} \qquad M_C = \frac{-Ph}{2} \cdot \frac{3k}{1+6k}$$

FIG. 8. STRESSES IN STIFF FRAMES WITH FIXED COLUMNS.

9



$-V_A = V_D = \frac{rc^2ks}{l(1+6k)}$ $\quad k = \frac{I_2}{I_1}\cdot\frac{h}{l}$ $\quad s = \frac{c}{h}$

$H_D = \frac{rcs^2}{8}\cdot\frac{4(1+k)-s(1+2k)}{2+k}$ $\quad H_A = rc - H_D$

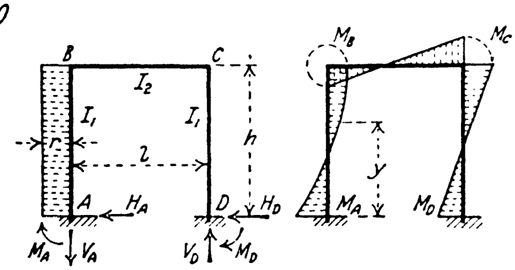
$M_A = \frac{-rc^2s}{24}\left[\frac{12}{s} - \frac{4(3+2k)-3s(1+k)}{2+k} - \frac{12k}{1+6k}\right]$

$M_D = \frac{-rc^2s}{24}\left[\frac{4(3+2k)-3s(1+k)}{2+k} - \frac{12k}{1+6k}\right]$

$-M_y = M_A + H_A y - \frac{ry^2}{2}$ $\quad M_E = -M_A - H_A c + \frac{rc^2}{2}$

$-M_B = M_A + H_D h - \frac{rc^2}{2}$ $\quad M_C = -M_D - H_D h$

10



$-V_A = V_D = \frac{rh^2k}{l(1+6k)}$ $\quad k = \frac{I_2}{I_1}\cdot\frac{h}{l}$

$H_D = \frac{rh}{8}\cdot\frac{3+2k}{2+k}$ $\quad H_A = rh - H_D$

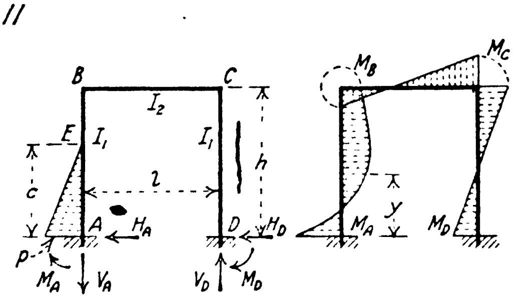
$M_A = \frac{-rh^2}{24}\left(12 - \frac{9+5k}{2+k} - \frac{12k}{1+6k}\right)$

$M_D = \frac{-rh^2}{24}\left(\frac{9+5k}{2+k} - \frac{12k}{1+6k}\right)$

$-M_y = M_A + H_A y - \frac{ry^2}{2}$

$-M_B = M_A + H_A h - \frac{rh^2}{2}$ $\quad M_C = -M_D - H_D h$

11



$-V_A = V_D = \frac{pkc^2s}{4l(1+6k)}$ $\quad k = \frac{I_2}{I_1}\cdot\frac{h}{l}$ $\quad s = \frac{c}{h}$

$H_D = \frac{pcs^2}{40}\cdot\frac{5(1+k)-s(1+2k)}{2+k}$ $\quad H_A = \frac{pc}{2} - H_D$

$M_A = \frac{-pc^2s}{120}\left[\frac{20}{s} - \frac{5(3+2k)-3s(1+k)}{2+k} - \frac{15k}{1+6k}\right]$

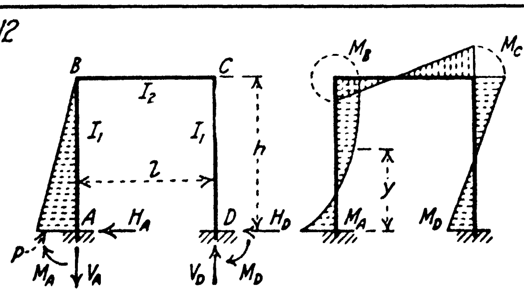
$M_D = \frac{-pc^2s}{120}\left[\frac{5(3+2k)-3s(1+k)}{2+k} - \frac{15k}{1+6k}\right]$

$-M_y = M_A + H_A y - \frac{py^2}{6c}(3c-y)$

$-M_E = M_A + H_A c - \frac{pc^2}{3}$ $\quad M_B = -M_A - H_D h + \frac{pc^2}{6}$

$M_C = -M_D - H_D h$

12



$-V_A = V_D = \frac{pkh^2}{4l(1+6k)}$ $\quad k = \frac{I_2}{I_1}\cdot\frac{h}{l}$

$H_D = \frac{ph}{40}\cdot\frac{4+3k}{2+k}$ $\quad H_A = \frac{ph}{2} - H_D$

$M_A = \frac{-ph^2}{120}\left(20 - \frac{12+7k}{2+k} - \frac{15k}{1+6k}\right)$

$M_D = \frac{-ph^2}{120}\left(\frac{12+7k}{2+k} - \frac{15k}{1+6k}\right)$

$-M_y = M_A + H_A y - \frac{py^2}{6h}(3h-y)$

$M_B = -M_A - H_D h + \frac{ph^2}{6}$ $\quad M_C = -M_D - H_D h$

FIG. 9. STRESSES IN STIFF FRAMES WITH FIXED COLUMNS.

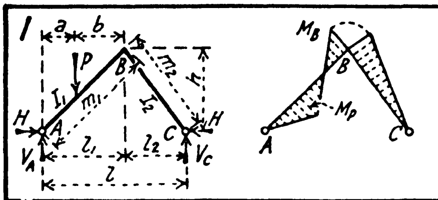
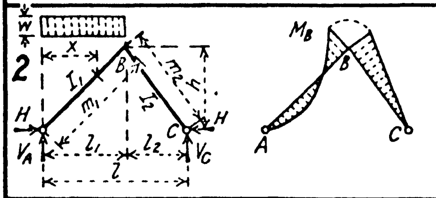
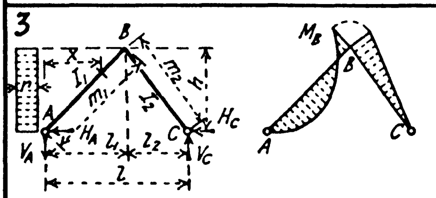
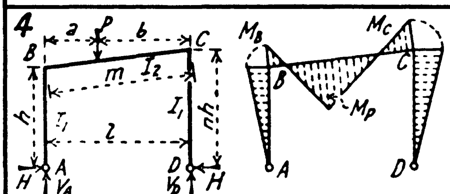
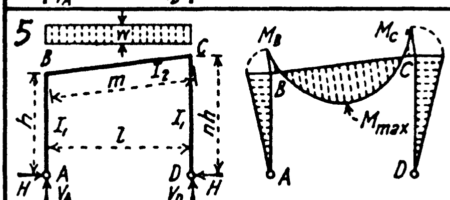
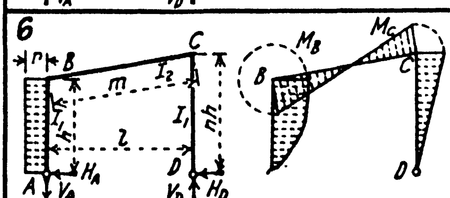
1		$V_A = \frac{P(l-a)}{l}$ $\quad k = \frac{I_2 m_1}{I_1 m_2}$ $\quad s = \frac{a}{l_1}$ $V_C = \frac{Pa}{l}$ $H = \frac{Pa}{2hl}\frac{[2l_2 + (3l - ls^2 - 2l_1)k]}{(1+k)}$ $M_P = V_A a - H\frac{ha}{l_1}$ $M_B = V_C l_2 - H_C h$	If $l_1 = l_2 = \frac{l}{2}$ and $k = 1$, $s = \frac{2a}{l}$ $H = \frac{Pa}{4h}(3 - s^2)$ If $a = l_1 = \frac{l}{2}$ and $k = 1$, $s = 1$ $H = \frac{Pl}{4h}$
2		$V_A = \frac{wl_1}{2l}(l + l_2)$ $\quad k = \frac{I_2 m_1}{I_1 m_2}$ $V_C = \frac{wl_1^2}{2l}$ $H = \frac{wl_1^2}{8hl}\frac{[4l_2 + k(l + 4l_2)]}{(1+k)}$ $M_X = V_A x - H\frac{hx}{l_1} - \frac{wx^2}{2}$ $M_B = V_C l_2 - H_C h$	If $l_1 = l_2 = \frac{l}{2}$ and $k = 1$ $H = \frac{5wl^2}{64h}$ Both sides loaded $l_1 = l_2 = \frac{l}{2}$ and $k = 1$ $H = \frac{5wl^2}{32h}$
3		$-V_A = V_C = \frac{rh^2}{2l}$ $\quad k = \frac{I_2 m_1}{I_1 m_2}$ $H_C = \frac{rh}{8l}\frac{[4l_2 + k(l + 4l_2)]}{(1+k)}$ $H_A = rh - H_C$ $M_x = H_A\frac{hx}{l_1} - V_A x - \frac{rh^2 x^2}{2l_1^2}$ $M_B = V_C l_2 - H_C h$	If $l_1 = l_2 = \frac{l}{2}$ and $k = 1$, $H_C = \frac{5rh}{16}$ $H_A = \frac{11rh}{16}$ $M_B = \frac{-rl^2}{16}$
4		$V_A = \frac{Pb}{l}$ $\quad V_D = \frac{Pa}{l}$ $\quad k = \frac{I_2 h}{I_1 m}$ $H = \frac{Pab[l(2+n) + a(n-1)]}{2hl^2[1 + n + n^2 + k(1 + n^3)]}$ $M_B = -Hh$ $\quad M_C = -nHh$ $M_{max} = \frac{Pab}{l} - H[h + \frac{a}{l}(nh - h)]$	If $a = b = \frac{l}{2}$ $V_A = V_D = \frac{P}{2}$ $H = \frac{3Pl}{16h}\frac{(1+n)}{\beta}$ $\beta = 1 + n + n^2 + k(1 + n^3)$
5		$V_A = V_D = \frac{wl}{2}$ $\quad k = \frac{I_2 h}{I_1 m}$ $H = \frac{wl^2}{8h}\frac{(1+n)}{[1 + n + n^2 + k(1 + n^3)]}$ $M_B = -Hh$ $\quad M_C = -nHh$ $M_{max} = \frac{wl^2}{8} - Hh(\frac{1+n}{2})$	Horizontal uniform load on vert. proj. of roof $-V_A = V_D = \frac{rh^2(n^2 - 1)}{2l}$ $H_A = \frac{rh}{8}\frac{\alpha}{\beta}$ where $\alpha = 8n^3 k(n-1) + 7n^3 - 3n^2 - 3n - 1$ $\beta = 1 + n + n^2 + k(1 + n^3)$ $H_D = rh(n-1) - H_A$ $M_B = H_A h$ $M_C = -nh[rh(n-1) - H_A]$
6		$-V_A = V_D = \frac{rh^2}{2l}$ $\quad k = \frac{I_2 h}{I_1 m}$ $H_D = \frac{rh}{8}\frac{(5k + 4 + 2n)}{[1 + n + n^2 + k(1 + n^3)]}$ $H_A = rh - H_D$ $M_B = H_A h - \frac{rh^2}{2}$ $M_C = -nHh$	

FIG. 10. STRESSES IN STIFF FRAMES.

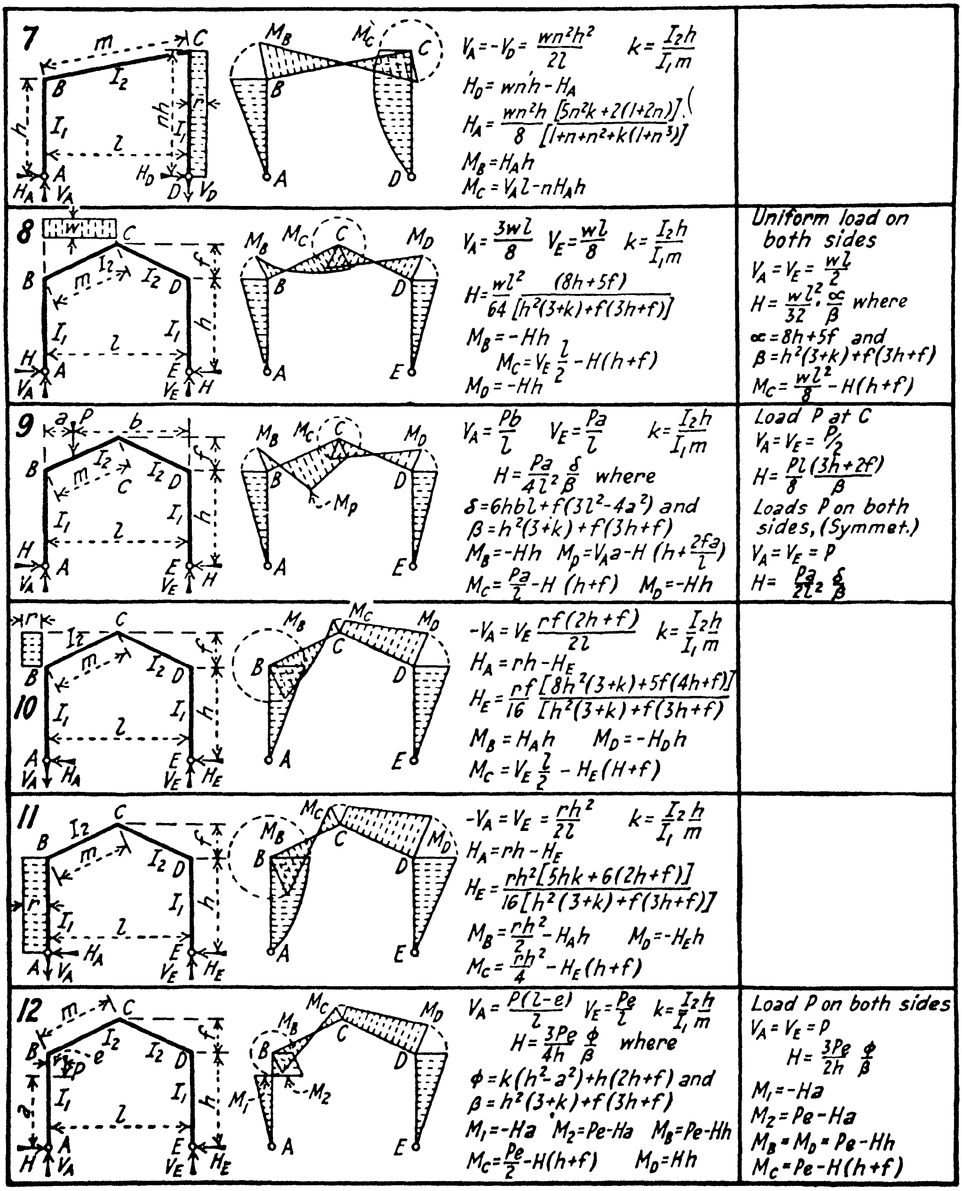
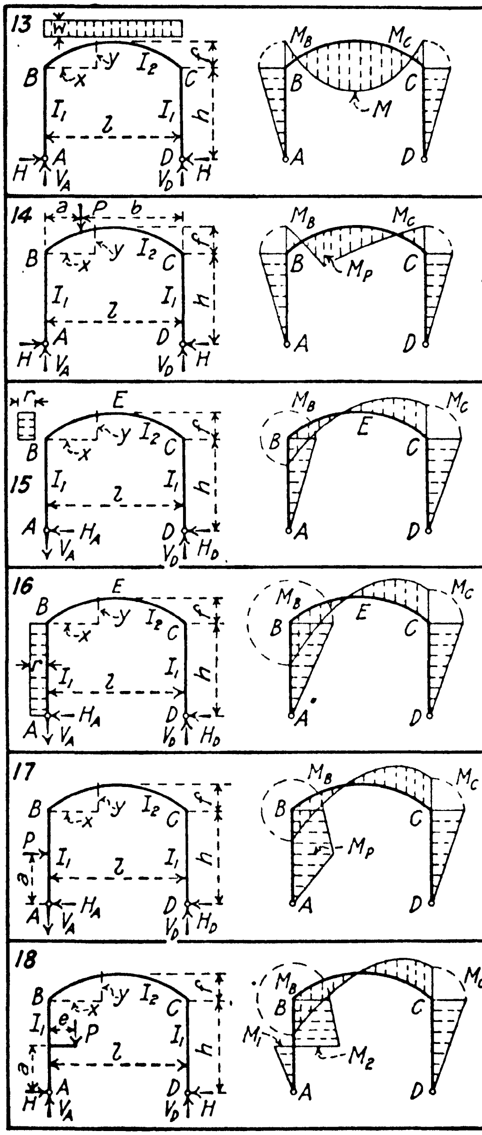


FIG. 11. STRESSES IN STIFF BUILDING FRAMES.



13. $V_A = V_D = \frac{wl}{2}$ $k = \frac{I_2 h}{I_1 l}$ $y = \frac{4fx}{l^2}(l-x)$

$$H = \frac{wl^2}{4}\cdot\frac{(5h+4f)}{[5h^2(2k+3)+4f(5h+2f)]}$$

$M_B = M_C = -Hh$

$M_x = \frac{wx}{2}(l-x) - H(h+y)$ $M = \frac{wl^2}{8} - H(h+f)$

14. $V_A = \frac{Pb}{l}$ $V_D = \frac{Pa}{l}$ $k = \frac{I_2 h}{I_1 l}$ $y = \frac{4fx}{l^2}(l-x)$

$$H = \frac{5Pa}{2l^3}\cdot\frac{(3bhl^2+2fl^3-4fa^2l+2fa^3)}{[5h^2(2k+3)+4f(5h+2f)]}$$

$M_B = M_C = -Hh$ $M_x = M - H(h+y)$ where M = moment in simple beam BC

$M_P = \frac{Pab}{l} - H(h+y)$

15. $-V_A = V_D = \frac{rf}{2l}(2h+f)$ $k = \frac{I_2 h}{I_1 l}$ $y = \frac{4fx}{l^2}(l-x)$

$$H_D = \frac{rf}{28}\cdot\frac{[70h^2(2k+3)+f(224h+64f)]}{[5h^2(2k+3)+4f(5h+2f)]}$$

$H_A = rf - H_D$ $M_B = H_A h$ $M_C = -H_D h$

$M_E = \frac{V_D l}{2} - H_D(h+f)$

16. $-V_A = V_D = \frac{rh^2}{2l}$ $k = \frac{I_2 h}{I_1 l}$ $y = \frac{4fx}{l^2}(l-x)$

$$H_D = \frac{5rh^2}{8}\cdot\frac{[h(5k+6)+4f]}{[5h^2(2k+3)+4f(5h+2f)]}$$

$H_A = rh - H_D$ $M_B = \frac{rh^2}{2} - H_D h$ $M_C = -H_D h$

$M_E = \frac{rh^2}{4} - H_D(h+f)$

17. $-V_A = V_D = \frac{Pa}{l}$ $k = \frac{I_2 h}{I_1 l}$ $y = \frac{4fx}{l^2}(l-x)$

$$H_D = \frac{5Pa}{2h}\cdot\frac{[3h^2(k+1)+2fh-a^2k]}{[5h^2(2k+3)+4f(5h+2f)]} \qquad H_A = P - H_D$$

$M_P = H_A a$ $M_B = Pa - H_D h$ $M_C = -H_D h$

If $a = h$, $H_D = \frac{5P}{2}\cdot\frac{[h^2(2k+3)+2fh]}{[5h^2(2k+3)+4f(5h+2f)]}$

18. $V_A = \frac{P(l-e)}{l}$ $V_D = \frac{Pa}{l}$ $k = \frac{I_2 h}{I_1 l}$ $y = \frac{4fx}{l^2}(l-x)$

$$H = \frac{5Pe}{2h}\cdot\frac{[3k(h^2-a^2)+h(3h+2f)]}{[5h^2(2k+3)+4f(5h+2f)]}$$

$M_1 = -Ha$ $M_2 = Pe - Ha$

$M_B = Pe - Hh$ $M_C = -Hh$

FIG. 12. STRESSES IN STIFF FRAMES.

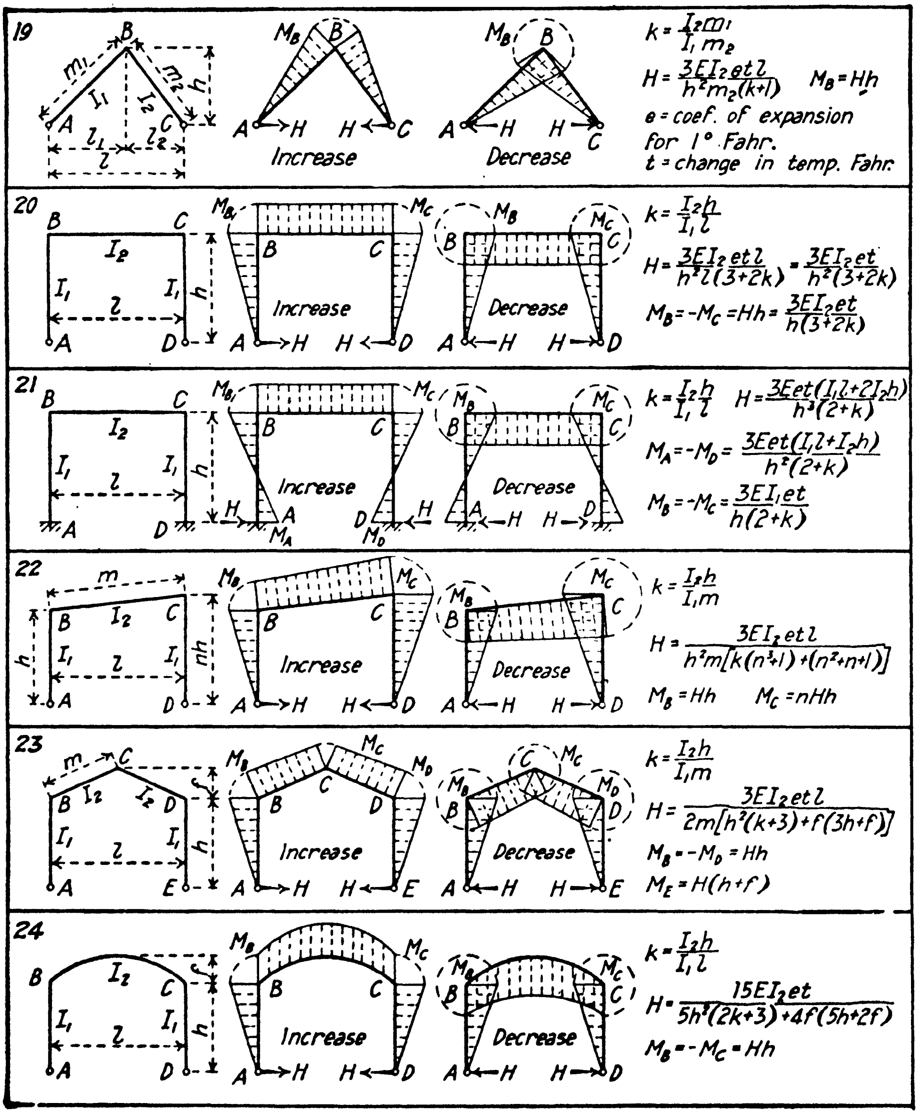
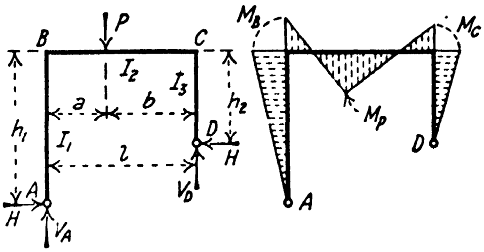


FIG. 13. TEMPERATURE STRESSES IN STIFF FRAMES.



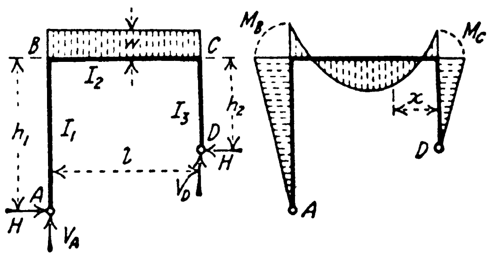
$$V_A = P\cdot\frac{b}{l} + H\cdot\frac{h_1-h_2}{l} \qquad k_1 = \frac{I_2}{I_1}\cdot\frac{h_1}{l}$$

$$V_D = P\cdot\frac{a}{l} - H\cdot\frac{h_1-h_2}{l} \qquad k_2 = \frac{I_2}{I_3}\cdot\frac{h_2}{l}$$

$$H = \frac{3abP(ah_1+bh_2)}{2l^2(k_1h_1^2+k_2h_2^2)+3ah_1^2(l+b)+3bh_2^2(l+a)+3h_1h_2(a^2+b^2)}$$

$$M_B = -Hh_1 \qquad M_C = -Hh_2$$

$$M_P = +V_A a - Hh_1 = V_D b - Hh_2$$



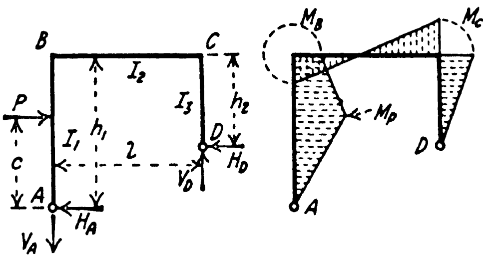
$$V_A = \frac{wl}{2} + H\cdot\frac{h_1-h_2}{l} \qquad k_1 = \frac{I_2}{I_1}\cdot\frac{h_1}{l}$$

$$V_D = \frac{wl}{2} - H\cdot\frac{h_1-h_2}{l} \qquad k_2 = \frac{I_2}{I_3}\cdot\frac{h_2}{l}$$

$$H = \frac{wh_1l^2}{4(k_1h_1^2+k_2h_2^2)+6h_1(h_1+h_2)}$$

$$M_B = -Hh_1 \qquad M_C = -Hh_2$$

$$M_x = M_0 - \left[M_C + \frac{x}{l}(M_B - M_C)\right] \text{ where } M_0 = \frac{wx}{2}(l-x)$$

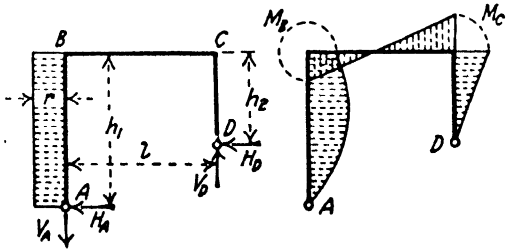


$$-V_A = V_D = \frac{P(h_2+c-h_1)+H_A(h_1-h_2)}{l}$$

$$H_D = P - H_A \qquad k_1 = \frac{I_2}{I_1}\cdot\frac{h_1}{l} \qquad k_2 = \frac{I_2}{I_3}\cdot\frac{h_2}{l}$$

$$H_A = P\cdot\frac{k_1(2h_1^3+c^3-3ch_1^2)+2k_2h_1h_2^2+3h_1^2(h_1-c+h_2)}{h_1^3(3+2k_1)+h_2(3h_1^2+2k_2h_1h_2)}$$

$$M_B = -H_Ah_1 + P(h_1-c) \qquad M_C = -H_Dh_2 \qquad M_P = -H_Ac$$



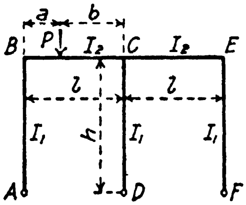
$$-V_A = V_D = \frac{rh_1\left(h_2-\frac{h_1}{2}\right)+H_A(h_1-h_2)}{l} \qquad k_1 = \frac{I_2}{I_1}\cdot\frac{h_1}{l}$$

$$H_D = rh_1 - H_A \qquad k_2 = \frac{I_2}{I_3}\cdot\frac{h_2}{l}$$

$$H_A = rh_1\cdot\frac{3k_1h_1^2+8k_2h_2^2+6h_1(h_1+2h_2)}{8(k_1h_1^2+k_2h_2^2)+12h_1(h_1+h_2)}$$

$$M_B = -H_Ah_1 + \frac{rh_1^2}{2} \qquad M_C = -H_Dh_2$$

FIG. 14. STRESSES IN STIFF FRAMES.



$k=\frac{I_2 h}{I_1 l}$

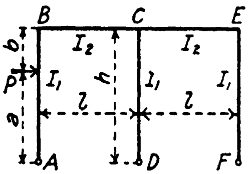
$$M_{BA}=-M_{BC}=\frac{+C_{BC}(10k+9)+C_{CB}(4k+3)}{4(k+1)(4k+3)}$$

$$M_{CB}=\frac{+(2k+3)[C_{BC}(2k+1)+C_{CB}(4k+3)]}{4(k+1)(4k+3)}$$

$$M_{CD}=-\frac{C_{BC}+C_{CB}}{2(k+1)}$$

$$M_{CE}=-\frac{C_{BC}(4k^2-3)+C_{CB}(4k+3)(2k+1)}{4(k+1)(4k+3)}$$

$$M_{EC}=-M_{EF}=+\frac{C_{BC}(2k+3)+C_{CB}(4k+3)}{4(k+1)(4k+3)}$$



$k=\frac{I_2 h}{I_1 l}$

$$M_{BA}=-M_{BC}=\frac{2kH_{BA}(16k+15)-M_A(4k+3)^2}{12(k+1)(4k+3)}$$

$$M_{CB}=-\frac{(2k+3)[2kH_{BA}-M_A(4k+3)]}{12(k+1)(4k+3)}$$

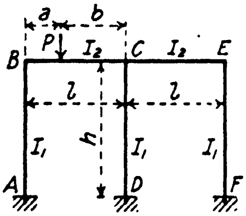
$$M_{CD}=-\frac{2kH_{BA}+M_A(2k+3)}{6(k+1)}$$

$$M_{CE}=\frac{2kH_{BA}(10k+9)+M_A(4k+3)(2k+3)}{12(k+1)(4k+3)}$$

$$M_{EC}=-M_{EF}=\frac{2kH_{BA}(8k+9)+M_A(4k+3)^2}{12(k+1)(4k+3)}$$

	Concentrated Load	Uniform Load	Symmetrical Load
FOR ALL CASES			
C_{BC}	$\frac{Pab^2}{l^2}$	$\frac{wl^2}{12}$	$\frac{\text{Moment Area}}{\text{Span}}$
C_{CB}	$\frac{Pa^2b}{l^2}$	$\frac{wl^2}{12}$	$\frac{\text{Moment Area}}{\text{Span}}$
H_{AB}	$\frac{Pab(h+b)}{h^2}$	$\frac{wh^2}{8}$	$\frac{3}{2}\,\frac{\text{Moment Area}}{\text{Span}}$
H_{BA}	$\frac{Pab(h+a)}{h^2}$	$\frac{wh^2}{8}$	$\frac{3}{2}\,\frac{\text{Moment Area}}{\text{Span}}$
C_{AB}	$\frac{Pab^2}{h^2}$	$\frac{wh^2}{12}$	$\frac{\text{Moment Area}}{\text{Span}}$
C_{BA}	$\frac{Pa^2b}{h^2}$	$\frac{wh^2}{12}$	$\frac{\text{Moment Area}}{\text{Span}}$

M_A = statical moment of horizontal loads on member AB about A. w = load per unit.



$k=\frac{I_2 h}{I_1 l}$

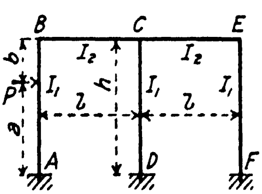
$$M_{BA}=-M_{BC}=\frac{C_{BC}(11k^2+15k+2)+4kC_{CB}(k+1)}{2(k+1)(6k^2+9k+1)}$$

$$M_{CB}=\frac{kC_{BC}(3k^2+8k+4)+C_{CB}(k+1)(6k^2+13k+2)}{2(k+1)(6k^2+9k+1)}$$

$$M_{CD}=-\frac{7kC_{BC}+2C_{CB}(4k+1)}{2(6k^2+9k+1)} \qquad M_{AB}=\frac{kC_{BC}(4k+5)+C_{CB}(k+1)(2k+1)}{2(k+1)(6k^2+9k+1)}$$

$$M_{CE}=-\frac{kC_{BC}(3k^2+k-3)+kC_{CB}(k+1)(6k+5)}{2(k+1)(6k^2+9k+1)} \qquad M_{DC}=-\frac{C_{BC}(5k+1)+4kC_{CB}}{2(6k^2+9k+1)}$$

$$M_{EC}=-M_{EF}=\frac{kC_{BC}(k+3)-4kC_{CB}(k+1)}{2(k+1)(6k^2+9k+1)} \qquad M_{FE}=-\frac{C_{BC}(2k^2+4k+1)-C_{CB}(k+1)(2k+1)}{2(k+1)(6k^2+9k+1)}$$



$k=\frac{I_2 h}{I_1 l}$

$$M_{BA}=-M_{BC}=-\frac{kC_{BA}(10k^2+15k+3)-2k(M_A-C_{AB})(k+1)^2}{2(k+1)(6k^2+9k+1)}$$

$$M_{CB}=-\frac{k(k+2)[C_{BA}(2k+1)-(M_A-C_{AB})(k+1)]}{2(k+1)(6k^2+9k+1)} \qquad M_{DC}=-\frac{C_{BA}(6k^2-3k-1)+2(M_A-C_{AB})(3k^2+6k+1)}{6(6k^2+9k+1)}$$

$$M_{CD}=-\frac{kC_{BA}(2k-3)+2k(M_A-C_{AB})(k+2)}{2(6k^2+9k+1)} \qquad M_{FE}=-\frac{C_{BA}(6k^3+9k^2-k-1)+(M_A-C_{AB})(k+1)(6k^2+9k+2)}{6(k+1)(6k^2+9k+1)}$$

$$M_{CE}=\frac{kC_{BA}(4k^2+4k-1)+k(M_A-C_{AB})(k+1)(k+2)}{2(k+1)(6k^2+9k+1)}$$

$$M_{EC}=-M_{EF}=\frac{kC_{BA}(2k^2+3k-1)+2k(M_A-C_{AB})(k+1)^2}{2(k+1)(6k^2+9k+1)}$$

$$M_{AB}=-\frac{C_{BA}(6k^3+27k^2+26k+2)}{6(k+1)(6k^2+9k+1)}-\frac{M_A(6k^2+9k+2)+C_{AB}(30k^2+45k+4)}{6(6k^2+9k+1)}$$

FIG. 15. STRESSES IN STIFF FRAMES.

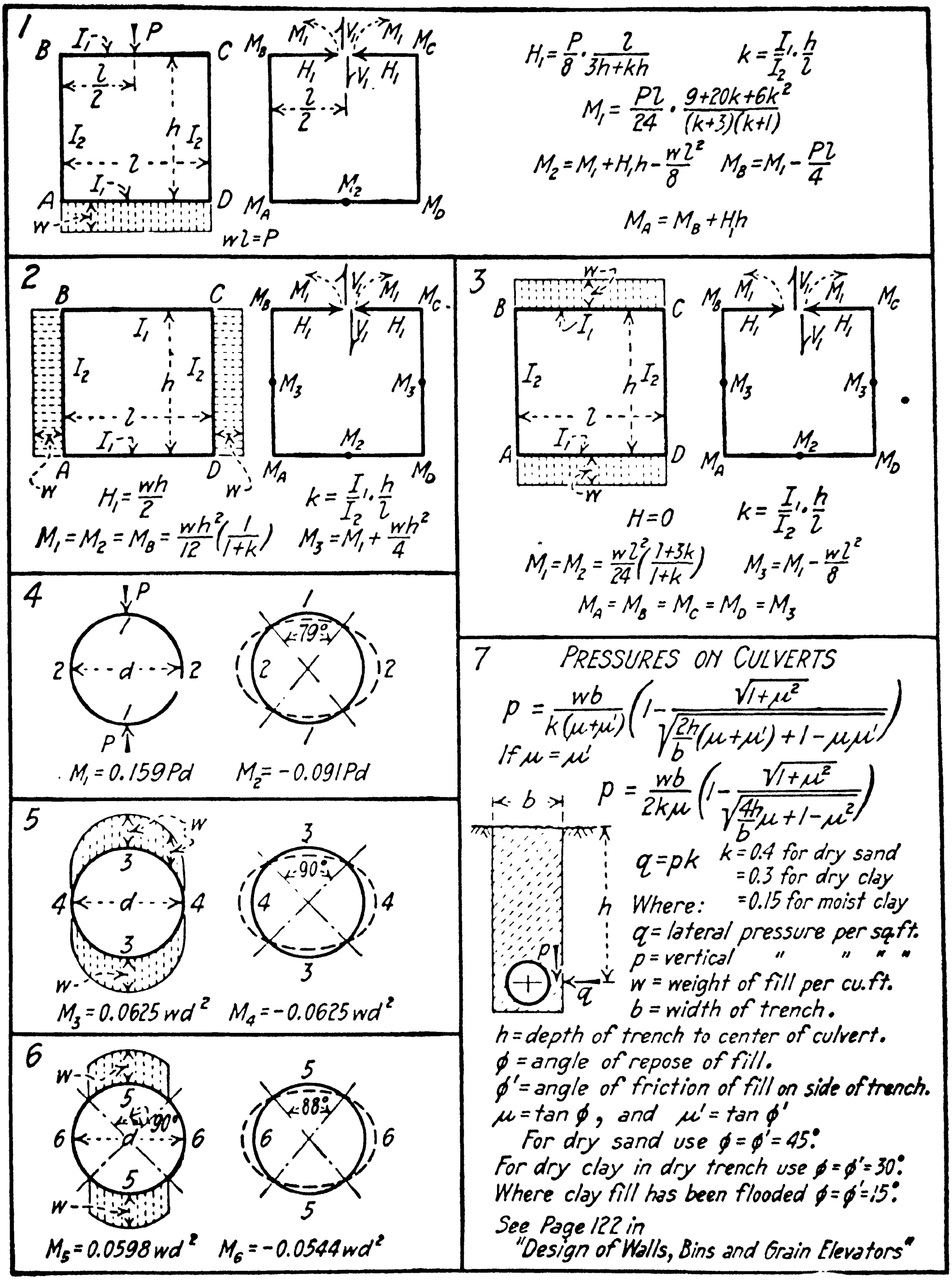


FIG. 16. STRESSES IN STIFF FRAMES.

CHAPTER XVII.

THE DESIGN OF STEEL DETAILS.

Introduction.—The design of any structure involves the design of the different members and the connections. In this chapter the design of the various steel details will be considered as fully and completely as the limited space permits. The design of the members and details of a steel structure are governed by the specifications for the particular structure. Reference will be made by section and page to the various specifications in this book.

MEMBERS IN TENSION.—Several different methods for making end connections of bars are shown in Fig. 1. Loop Bars, (*a*) Fig. 1, are used for lateral bracing on highway bridges, buildings and towers, with turnbuckles or sleeve nuts, to make them adjustable as shown in Tables 92 and 94. (All tables numbered with Arabic numerals are in Part II.) Clevises, (*b*) Fig. 1, are used to secure the ends of bars used as lateral bracing on highway bridges and on buildings. The pin may be either a cotter pin as shown in Table 96, or a bridge pin as shown in Table 95. Ordinary eye-bars, (*c*) Fig. 1, are used principally for lower chords and main ties on bridges. Data for eye-bars are given in Table 91. Counters are made of adjustable eye-bars as shown in Table 91. Bottom lateral plates or skew-backs, (*d*) Fig. 1, are used to secure the ends of bottom lateral rods

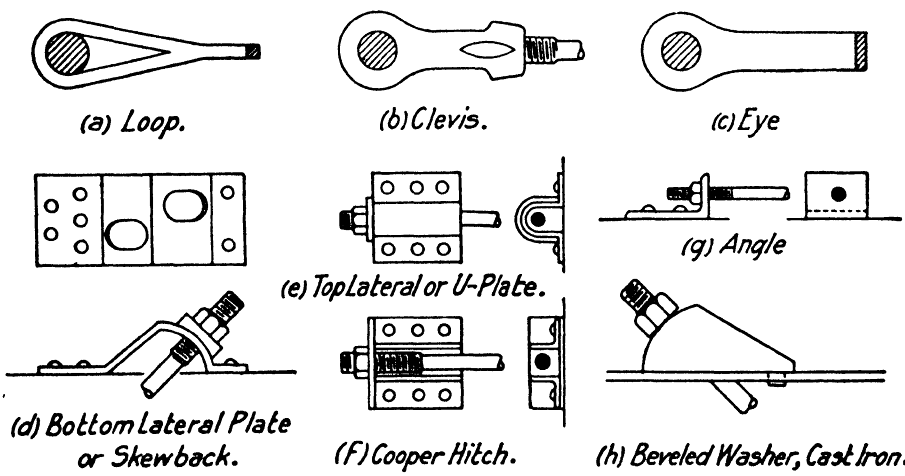


FIG. 1. DETAILS OF TENSION MEMBERS.

of highway bridges and are shown in Table 121. Top lateral plates or U-plates, (*e*) Fig. 1, are used for top lateral connections on highway bridges and for lateral bracing on buildings, highway bridges and towers, see Table 122. The Cooper hitch has the same uses as the top lateral plate. The angle as shown in (*g*) Fig. 1 is used for end connections for light bars in buildings and towers, see Table 120. Cast iron beveled washers, (*h*) Fig. 1, are used for end connections of diagonal bracing, see Table 120. The ends of bars should be upset as shown in Tables 89 and 90, so that the strength in the threads will be greater than the strength of the main body of the bar. The dimensions of tie rods for beams are shown in Table 105.

In selecting bars in tension the area is determined by the formula:

$$A = \frac{P}{f_t}$$

where A is the required area, P the total tension in the bar and f_t the allowable unit tensile stress. The following problems are given to illustrate the use of the tables in selecting the details for bars, etc.

Loop Bar.—Select a loop bar to carry a tensile stress of 48,000 lb., one end passing around a 3 in. pin and the other end around a 3½ in. pin, the center to center distance between pins being 30′ 0″.

References.—Specification § 8, p. 73; § 37, p. 76; § 48, p. 78; § 103, p. 83; § 118, p. 84; § 127, p. 84; § 37, p. 189; § 49, p. 190; § 61, p. 191; § 15, p. 257; § 36, p. 258; § 230, p. 445; § 8, p. 461; § 42, p. 463; § 28, p. 467.

Solution.—Using an allowable unit stress of f_t = 16,000 lb. per sq. in., the area required is,

$$A = \frac{P}{f_t} = \frac{48{,}000}{16{,}000} = 3.00 \text{ sq. in.}$$

A bar 1¾ in. square has an area of 3.06 sq. in. (Table 6), and a 2 in. round bar has an area of 3.14 sq. in. (Table 6). Either bar could be used. Using the 1¾ in. square bar the additional length required to pass around a 3 in. pin is 1′ 11″ (Table 92), and for a 3½ in. pin is 2′ 1″, making it necessary to add 4′ 0″ to the center to center distance of pins to obtain the total length of bar.

If a *turnbuckle* is used the upset required on a 1¾ in. square bar is 2½ in. in diameter and 5½ in. long (Table 89), requiring 4½ in. extra material to make each upset, or 9 in. for the two upsets. The weight of a turnbuckle for a 2½ in. screw is 25 lb. (Table 94). The clearance between the ends of the screws for all turnbuckles is 5 in. (Diagram at top of Table 92).

The total length and weight of the 1¾ in. square bar is therefore:

c. to c. of pins, less 5 in.,	= 29′ 7″	of 1¾ in. square bar,	@ 10.41 lb. per ft. (Table 6)	= 308.0 lb.
Material for 2 loops	= 4′ 0″	of 1¾ in. square bar,	@ 10.41 lb. per ft. (Table 6)	= 41.6 lb.
Material for 2 upsets	= 0′ 9″	of 1¾ in. square bar,	@ 10.41 lb. per ft. (Table 6)	= 7.8 lb.
One Turnbuckle			@ 25 lb. (Table 94)	= 25.0 lb.
Total Length	= 34′ 4″		Total Weight	= 382.4 lb.

If a *sleeve nut* is used, instead of a turnbuckle, its weight for a 2½ in. screw, is 19 lb. (Table 94). The clearance between the ends of the screws is 3 in. for all sleeve nuts (Diagram at the top of Table 92).

The total length and weight of 1¾ in. square bar when a sleeve nut is used is therefore:

c. to c. of pins, less 3 in.,	= 29′ 9″	of 1¾ in. square bar,	@ 10.41 lb. per ft. (Table 6)	= 309.8 lb.
Material for 2 loops	= 4′ 0″	of 1¾ in. square bar,	@ 10.41 lb. per ft. (Table 6)	= 41.6 lb.
Material for 2 upsets	= 0′ 9″	of 1¾ in. square bar,	@ 10.41 lb. per ft. (Table 6)	= 7.8 lb.
One sleeve nut			@ 19 lb. (Table 94)	= 19.0 lb.
Total Length	= 34′ 6″		Total Weight	= 378.2 lb.

Bar with Clevises.—Select a bar to carry a tensile stress of 48,000 lb., the ends to be held by clevises, the distance center of pins being 12′ 0″.

References.—Same as for loop bar, also, § 118, p. 84; § 19, p. 135; § 56, p. 285; § 88, p. 287.

Solution.—Using an allowable unit stress of f_t = 16,000 lb. per sq. in., the area required is,

$$A = \frac{P}{f_t} = \frac{48{,}000}{16{,}000} = 3.00 \text{ sq. in.}$$

A bar 1¾ in. square has an area of 3.06 sq. in. (Table 6), and a 2 in. round bar has an area of 3.14 sq. in. (Table 6). Either bar could be used. Using the 1¾ in. square bar a No. 6 clevis is required (Table 93).

The size of pin required by shear and moment can be obtained from the lower part of Table 93, and is a 2 in. pin if the forks are closed, or a 3 in. pin if the forks are used straight. The thickness of connection plate required by bearing when a 2 in. pin is used, is 48,000 ÷ (2.00 × 24,000) = 1.00 in., if a 3 in. pin is used the plate must be 48,000 ÷ (3.00 × 24,000) = 0.66 in.

The weight of the bar and two clevises is estimated as follows:

The length of the rod, allowing for clearance, etc., must be reduced by $A - \frac{1}{2}$ in. = 8 − ½ = 7½ in. (Table 93) at each end, or a total of 2 × 7½ = 1′ 3″. The diameter of upset for a 1¾ in. square bar is 2½ in., which requires 4½ in. material to make each upset (Table 89), or 9 in. for both upsets.

The total length and weight of 1¾ in. square bar is:

c. to c. of pins, less 1′ 3″,	= 10′ 9″ of 1¾ in. square bar,	@ 10.41 lb. per ft. (Table 6)	= 111.9 lb.
Material for 2 upsets	= 0′ 9″ of 1¾ in. square bar,	@ 10.41 lb. per ft. (Table 6)	= 7.8 lb.
Two No. 6 clevises		@ 26 lb. (Table 93)	= 52.0 lb.
Total Length	= 11′ 6″	Total Weight	= 171.7 lb.

Eye-Bar.—Select an eye-bar to carry a tensile stress of 190,000 lb., with an 8 in. pin at one end and a 6½ in. pin at the other end, the length center to center of pins being 25′ 0″.

References.—§ 37, p. 76; § 116, p. 84; § 172, p. 90; § 92, p. 193; § 141, p. 194; § 171, p. 196; § 14, p. 254; "Minimum Bar," p. 255; § 15, p. 257; § 36, p. 258; § 83, p. 261; § 162, p. 266; § 38, p. 284; § 84, p. 287; § 139, p. 290; § 243, p. 293; § 282, p. 295.

Solution.—Using an allowable unit stress of f_t = 16,000 lb. per sq. in., the area required is,

$$A = \frac{P}{f_t} = \frac{190{,}000}{16{,}000} = 11.87 \text{ sq. in.}$$

A bar 8 in. × 1½ in. has an area of 12.00 sq. in. (Table 1). From Table 91, the maximum thickness allowed for an 8 in. bar on a 6½ in. pin is 2 in., and the minimum is 1 in. (The value 6½ in. does not appear in the table but it is less than 7 in., which is the maximum pin which can be used if the die referred to is used.) For an 8 in. pin the maximum thickness is 2 in. and the minimum 1⅛ in. The bar selected satisfies these requirements as to thickness.

The extra length of bar required to form a head for a 6½ in. pin (die for 7 in. pin) is 2′ 8″ for ordering the bar, and 2′ 3″ for estimating the weight, and for an 8 in. pin 3′ 0″ and 2′ 6″, respectively (Table 91).

The total length and weight of eye-bar is therefore:

c. to c. of pins	= 25′ 0″ of 8 in. × 1½ in. bar.	@ 40.8 lb. per ft. (Table 2)	= 1020.0 lb.
Eye for 6½ in. pin	= 2′ 3″ of 8 in. × 1½ in. bar,	@ 40.8 lb. per ft.	= 91.8 lb.
Eye for 8 in. pin	= 2′ 6″ of 8 in. × 1½ in. bar,	@ 40.8 lb. per ft.	= 102.0 lb.
Total Length	= 29′ 9″	Total Gross Weight	= 1213.8 lb.

The weight which must be deducted for pin holes (Table 6) is,

Pin hole for 6½ in. pin is 1.5 ÷ 12 × 112.8	= 14.1 lb.
Pin hole for 8 in. pin is 1.5 ÷ 12 × 171.0	= 21.4 lb.
Total weight to be deducted	= 35.5 lb.

The net weight of the eye-bar is then 1213.8 − 35.5 = 1178.3 lb.

For the design of an eye-bar subject to flexure due to its own weight, see "Combined Flexure and Direct Stress" in this chapter.

Angle in Tension.—Select an angle to carry a tensile stress of 40,000 lb., using ¾ in. rivets.

References.—§ 37, p. 76; § 42, p. 77; § 46, p. 77; § 47, p. 77; § 48, p. 78; § 98, p. 83; § 102, p. 83; § 114, p. 84; § 22, p. 129; § 4, p. 131; § 12, p. 133; § 37, p. 189; § 43, p. 189; § 60, p. 191; § 79, p. 192; § 26, p. 254; § 45, p. 254; "Fastening Angles," p. 255; § 15, p. 257; § 21, p. 257; § 74, p. 260; § 38, p. 284; § 55, p. 285; § 62, p. 286; § 77, p. 287; § 232, p. 443; § 8, p. 461.

Solution.—If fastened by both legs as in Fig. 2 the load may be considered as axial and the required net area, using an allowable unit stress of f_t = 16,000 lb. per sq. in., is

$$A = \frac{P}{f_t} = \frac{40,000}{16,000} = 2.50 \text{ sq. in.}$$

Try one angle 4″ × 4″ × ⅜″. Gross area = 2.86 sq. in. (Table 23 or Table 25). Net area, deducting one ⅞ in. hole for a ¾ in. rivet = 2.86 − .33 = 2.53 sq. in. (Table 116). This angle will satisfy the conditions. This result can be obtained directly from Table 29.

If the angle is fastened by one leg as in Fig. 3, the load will be eccentric and the problem more difficult. An approximate solution is to consider only the area of the attached leg as effective. The solution would then be, as before

$$A = \frac{P}{f_t} = \frac{40,000}{16,000} = 2.50 \text{ sq. in.}$$

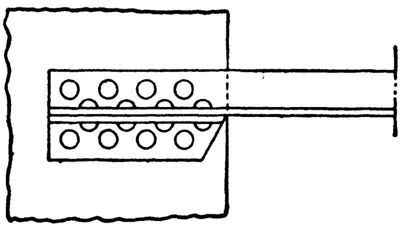


FIG. 2. ANGLE CONNECTED BY BOTH LEGS.

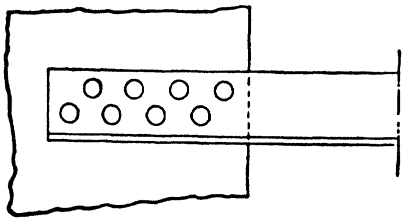


FIG. 3. ANGLE CONNECTED BY ONE LEG.

Try one angle 6″ × 4″ × ½″ with 6 in. leg attached. Gross area of 6 in. leg = 6 × ½ = 3.00 sq. in., net area = 3.00 − .44 = 2.56 sq. in., which will satisfy the conditions.

Built-up Tension Member.—Design a built-up member to carry a tensile stress of 390,000 lb., using ⅞ in. rivets.

References.—§ 37, p. 76; § 47, p. 77; § 102, p. 83; § 111, p. 84; § 114, p. 84; § 5, p. 131; § 12, p. 133; § 37, p. 189; § 44, p. 189; § 75, p. 192; § 14 and § 26, p. 254; § 28, p. 258; § 52, p. 259; § 38, p. 284; § 76 to § 83, p. 286; § 138, p. 290; § 9, p. 461; § 11, p. 464.

Solution.—Using an allowable unit stress of f_t = 16,000 lb. per sq. in., the net area required is,

$$A = \frac{P}{f_t} = \frac{390,000}{16,000} = 24.4 \text{ sq. in.}$$

Try 4 angles 3½″ × 3½″ × ½″ and 2 plates 18 in. × ½ in., as shown in Fig. 4. Gross area = 18.00 + 13.00 = 31.00 sq. in. Referring to Fig. 4, it will be seen that the section *n–n* is the least section in the body of the member and that four rivet holes should be deducted from each side to obtain the net section, giving a net area of 31.00 − 4.00 − 2.00 = 25.00 sq. in., 4.00 sq. in. being the area of holes in the plates and 2.00 sq. in. being the area of holes in the angles, deducting 1 in. holes for ⅞ in. rivets. This section has sufficient area, 24.4 sq. in. being required.

If the ends of the members are to be riveted they should be designed as outlined under "Riveted Connections and Joints" in this chapter.

If the ends are to be pin-connected they may be designed as follows. Assume that 5½ in. pins are to be used at each end. The bearing area required allowing a unit stress of 24,000 lb. per sq. in., is 390,000 ÷ 24,000 = 16.2 sq. in. This requires a total thickness of plates of 16.2 ÷ 5.5 = 2.95 in., or 1.48 in. on each side. The web plates are ½ in., the fill plates must be at least ½ in., the thickness of the angles being ½ in., and using ½ in. outside plates the total thickness of plates is 1.50 in., which satisfies the conditions, 1.48 in. being required.

The net area through the pin hole (section m–m) must be 25 per cent in excess of the net area of the body of the member according to a common specification. It will probably be necessary to deduct the area of the pin hole and two rivet holes on each side, the rivet holes being so near the section m–m, see Fig. 4. The gross area through the pin hole is, web plates $2 \times 18 \times \frac{1}{2} = 18.00$ sq. in., angles $4 \times 3.25 = 13.00$ sq. in., fill plate $2 \times 11 \times \frac{1}{2} = 11.00$ sq. in., outside plate $2 \times 17 \times \frac{1}{2} = 17.00$ sq. in. making a total gross area of 59.00 sq. in. The net area is $59.00 - 2 \times 5.5 \times 1.5 - 4 \times 1 \times 1\frac{1}{2} = 36.5$ sq. in. The required net area through the pin hole is $1.25 \times 25.00 = 31.3$ sq. in.

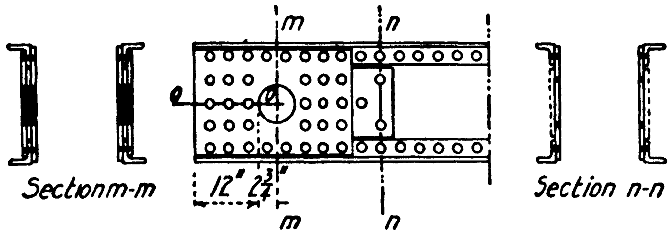


FIG. 4. RIVETED TENSION MEMBER.

The net area back of the pin hole parallel with the axis of the member (section o–o) must not be less than the net area in the body of the member (section n–n) = 25.0 sq. in. The total thickness of the metal at this section is 1.50 in. for each side. Therefore the net length back of the pin must be $25.00 \div 2 \times 1.50 = 8.33$ in. Assuming that not over three rivets will come in this section, the total length back of the pin hole must be at least $8.33 + 3.00 = 11.33$ in.

The number of rivets required and the size of pin plates is considered under "Riveted Connections and Joints."

Unriveted Pipe.—Design an unriveted iron pipe 12 in. in diameter to carry an internal pressure of 400 lb. per sq. in.

From Structural Mechanics, Chap. XVI (Formula 12a), $f = w \cdot D \div 2t$; and $t = w \cdot D \div 2f$, where t is the thickness of metal, w = unit internal pressure, D = diameter and f the allowable tensile stress which will be taken as 12,000 lb. per sq. in.

$$t = \frac{w \cdot D}{2f} = \frac{400 \times 12}{2 \times 12{,}000} = 0.20 \text{ in.}$$

MEMBERS IN COMPRESSION.—The design of compression members will be shown by several examples.

Single Angle Strut.—Select an angle to carry a compressive stress of 21,500 lb. The length center to center of connections is 6′ 0″, and both legs are to be fastened at the ends, Fig. 2.

References.—§ 36, p. 76; § 44, p. 77; § 48, p. 78; § 105, p. 83; § 5, p. 131; § 38, p. 189; § 60, p. 191; § 67, p. 191; § 45, p. 254; § 16, p. 257; § 20, p. 257; p. 267; § 38, p. 284; § 57, p. 285; § 231, p. 445.

Solution.—Using $f_c = 16{,}000 - 70\, l/r$ lb. per sq. in., as the allowable unit stress and 125 as the maximum value for the ratio l/r, the minimum value for r is as follows:

$$l/r = 125, \text{ or } r = \frac{l}{125} = \frac{6 \times 12}{125} = 0.58 \text{ in.}$$

Any 3″ × 3″ angle will satisfy the requirement for l/r (Table 23). The allowable unit stress will then be $16{,}000 - 70 \times \frac{72}{.58} = 7{,}300$ lb. per sq. in. The area required will be

$$A = \frac{P}{f_c} = \frac{21{,}500}{7{,}300} = 2.95 \text{ sq. in.}$$

The area of one angle 3″ × 3″ × 9/16″ is 3.06 sq. in., which is sufficient.

Many other angles might be chosen but in no case could an angle smaller than 3″ × 3″ be used, for the requirement for l/r would not be satisfied. Larger angles will give lighter sections and be more rigid. Any angle 3½″ × 3½″ has a radius of gyration, r, of about 0.69 (Table 23), giving an l/r of about 104, and an allowable unit stress of about 8,700 lb. per sq. in. and requiring an area of 2.47 sq. in., which would be provided by one angle 3½″ × 3½″ × ⅜″. The minimum angle satisfying the l/r requirement is found as a guide in the selection of sections but is rarely a satisfactory section, except for long members with low stresses such as lateral bracing. Table 41, Part II, gives the safe loads for single angle struts fastened by both legs.

If the angle is fastened by one leg only as in Fig. 3, the load is eccentric and the problem is more difficult. An approximate solution is to consider only the area of the attached leg as effective. As before the least radius of gyration must be not less than 0.58 in., which corresponds to an allowable unit stress of 7,300 lb. per sq. in., requiring the area of the attached leg to be at least 2.95 sq. in. The requirement for radius of gyration would be satisfied by any 3½″ × 3″ angle, but to provide 2.95 sq. in. of area if attached by the 3½ in. leg the thickness would have to be 2.95 ÷ 3.50 = 0.85 in. requiring a 3½″ × 3″ × ⅞″ angle, which is a very poor section and would be much heavier than a section with longer legs to satisfy the same conditions, and much less rigid. The least radius of gyrations of any 5″ × 3½″ angle is about 0.76 in. (Table 24), and the allowable unit stress will be

$$f_c = 16{,}000 - 70\,l/r = 16{,}000 - 70 \times \frac{72}{0.76} = 9{,}370 \text{ lb. per sq. in.,}$$

requiring an area of the attached leg of

$$A = \frac{P}{f_c} = \frac{21{,}500}{9{,}370} = 2.30 \text{ sq. in.}$$

which would be provided by a 5″ × 3½″ angle of thickness equal to $\frac{2.30}{5} = .46$ in. An angle 5″ × 3½″ × ½″ could be used with the 5 in. leg attached.

Double Angle Strut.—The member a–b Fig. 5 is to consist of two angles back to back separated by ⅜ in. connection plates at the ends and washers ⅜ in. thick in the body of the member. Design for a compressive stress of 50,000 lb.

References.—§ 36, p. 76; § 41, p. 77; § 48, p. 78; § 105, p. 83; § 5, p. 131; § 38, p. 189; § 67, p. 191; § 16, p. 257; § 20, p. 257; § 38, p. 284; § 49, p. 285; § 231, p. 445; § 10, p. 461.

Solution.—Using $f_c = 16{,}000 - 70\,l/r$ lb. per sq. in. as the allowable unit stress, and 125 as the maximum value for the ratio l/r, the minimum value for r is found as follows

$$l/r = 125, \text{ or } r = \frac{l}{125} = \frac{8 \times 12}{125} = 0.77 \text{ in.}$$

The lengths about axes X–X and Y–Y are equal, so that for a well designed member the radii of gyration about the two axes should be as nearly equal as practicable. This condition is satisfied by using angles with unequal legs, short legs turned out.

A member composed of two 2½″ × 2″ angles, ⅜ in. back to back, with short legs turned out will have a least radius of gyration of about 0.78 in. (Table 40), the value for axis X–X being about 0.78 in. and Y–Y about 0.95 in. The allowable unit stress is then $f_c = 16{,}000 - 70\,l/r = 16{,}000 - 70 \times \frac{8 \times 12}{0.78} = 7{,}390$ lb. per sq. in., requiring an area of

$$A = \frac{P}{f_c} = \frac{50{,}000}{7{,}390} = 6.76 \text{ sq. in.}$$

This area cannot be supplied by two 2½″ × 2″ angles, but even though it could, larger angles would be more economical as well as more rigid. The minimum angle satisfying the l/r

requirement is found so as to guide in the selection of angles but is rarely a satisfactory section, except for a long member with low stresses, such as lateral bracing.

Try two angles 4" × 3" with the short legs turned out, 3/8 in. back to back. From Table 40 it is seen that for any thickness the least radius of gyration will be about the axis $X-X$, and will be about 1.26 in., giving an allowable unit stress of $f_c = 16{,}000 - 70 \times \frac{8 \times 12}{1.26} = 10{,}670$ lb. per sq. in., which requires an area of 50,000 ÷ 10,670 = 4.68 sq. in. The area of 2 angles 4" × 3" × 3/8" = 4.96 sq. in., which will satisfy the conditions. If the estimated radius of gyration does not agree closely enough with the actual radius of gyration, another calculation should be made, but this is not often necessary.

The spacing of the washers should be such that the l/r of one angle between the washers is not greater than the l/r for the whole member, or $l/r = \frac{8 \times 12}{1.26} = 76.2$, $l = 76.2 \times .64 = 48.7$ in., 0.64 being the least radius of gyration of one angle 4" × 3" × 3/8" (Table 24). One washer in the center will be sufficient.

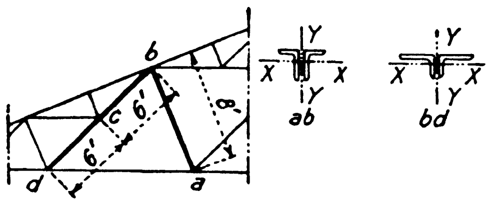


FIG. 5. DOUBLE ANGLE STRUT.

If lengths about the two axes are different, as is often the case in roof trusses and portals, the greatest value for l/r should be used, the corresponding length and radius of gyration being taken; for example in designing the member b–d, Fig. 5, as a strut the length corresponding to the axis $Y-Y$ is 12′ 0″, and to the axis $X-X$ is 6′ 0″. To make an efficient member the long legs should be turned out and r_y should be equal to $2 \times r_x$.

The minimum allowable values of r_x and r_y are found as follows,

$$l/r = 125, \quad r_x = \frac{l_x}{125} = \frac{6 \times 12}{125} = 0.58 \text{ in.};$$

$$r_y = \frac{l_y}{125} = \frac{12 \times 12}{125} = 1.15 \text{ in.}$$

From Table 39 it is seen that any 2½" × 2" angle with long legs turned out and 3/8 in. back to back is the smallest angle which will satisfy the requirements for l/r, $r_x = 0.58$ in. and $r_y = 1.26$ in. (approx.). The values for l/r are 124 and 114, respectively, 124 being the greater. The allowable unit stress is then

$$f_c = 16{,}000 - 70 \times 124 = 7{,}320 \text{ lb. per sq. in.}$$

If the stress in b–c is the same as that in c–d, 19,000 lb. compression, the required area is,

$$A = \frac{P}{f_c} = \frac{19{,}000}{7{,}320} = 2.60 \text{ sq. in.}$$

which will be taken by 2 angles 2½" × 2" × 5/16", having $r_x = 0.58$ in., and $r_y = 1.26$ in. (Table 39). If the stresses in b–c and c–d are not equal proceed as above and design for the maximum. The spacing of the washers should not be greater than, $l = 124 \times 0.42 = 52.1$ in., 0.42 in. being the least radius of gyration of one angle 2½" × 2" × 5/16".

If the controlling stress were 38,000 lb. compression, the required area for $2\frac{1}{2}'' \times 2''$ angles would be

$$A = \frac{P}{f_c} = \frac{38{,}000}{7{,}320} = 5.20 \text{ sq. in.}$$

which could not be supplied by two $2\frac{1}{2}'' \times 2''$ angles, so that two $3\frac{1}{2}'' \times 3''$ angles will be used for which, $r_x = 0.90$ and $r_y = 1.66$ for $\frac{3}{8}$ in. back to back, the values of l/r are $\frac{6 \times 12}{0.90} = 80$ and $\frac{12 \times 12}{1.66} = 86.8$, respectively, and the allowable unit stress is, $f_c = 16{,}000 - 70 \times 86.8 = 9{,}930$ lb. per sq. in., requiring an area of $A = 30{,}000 \div 9{,}930 = 3.83$ sq. in., which will be furnished by two angles $3\frac{1}{2}'' \times 3'' \times 5/16''$. The spacing of the washers should not be greater than, $l = 86.8 \times 0.63 = 54.6$ in., 0.63 in. being the least radius of gyration of one angle $3\frac{1}{2}'' \times 3'' \times 5/16''$. These results may be obtained by the use of Tables 43, 44 and 45, from which it is seen that the allowable stress in a member composed of two angles $3\frac{1}{2}'' \times 3'' \times 5/16''$ about axis 1–1 (Y–Y), the length being 12′ 0″, is 38,000 lb., and about axis 2–2 (X–X), the length being 6′ 0″, is 40,000 lb., and the allowable load will be 38,000 lb.

Two Angles Starred.—Design a member consisting of two angles starred, as in Fig. 6, to carry a compressive stress of 30,000 lb., the length to be 15′ 0″ center to center of connections.

Solution.—Using 125 as the maximum value of l/r, and $f_c = 16{,}000 - 70\, l/r$ lb. per sq. in. as the allowable unit stress, the minimum allowable value of r is found to be

$$l/r = 125,\quad r = \frac{l}{125} = \frac{15 \times 12}{125} = 1.44 \text{ in.}$$

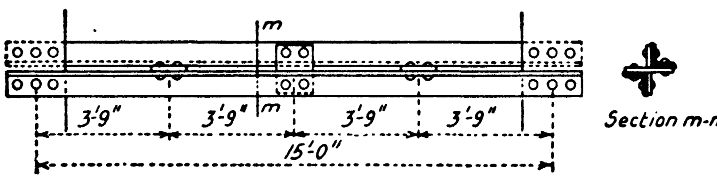


FIG. 6. TWO ANGLES STARRED.

From Table 67 it is seen that $4'' \times 4''$ angles are the smallest equal leg angles that can be used, and that r will be about 1.56 in., and the allowable unit stress is

$$f_c = 16{,}000 - 70 \times \frac{15 \times 12}{1.56} = 7{,}920 \text{ lb. per sq. in.,}$$

which requires an area of

$$A = \frac{P}{f_c} = \frac{30{,}000}{7{,}920} = 3.79 \text{ sq. in.}$$

The area of two angles $4'' \times 4'' \times \frac{1}{4}''$ is 3.88 sq. in., and $r = 1.57$ in., which will satisfy the conditions. The batten plates must have a spacing of not more than

$$l = \frac{15 \times 12}{1.57} \times 0.79 = 75 \text{ in.} = 6'\ 3'';$$

the value of 0.79 in. being the least radius of gyration for one angle $4'' \times 4'' \times \frac{1}{4}''$ (Table 23). Convenience in detailing may make it advisable to make l much less than 6′ 3″. A spacing of 3′ 9″ was used as shown in Fig. 6.

Plate and Angle Column.—Design a plate and angle column, Fig. 7, to carry an axial load of 340,000 lb., the unsupported length being 16′ 0″.

References.—§ 36, p. 76; § 44, p. 77; § 106, p. 83; § 5, p. 131.

Solution.—A section with a 12 in. web plate and two 14 in. flange plates will be assumed. The angles will be spaced 12½ in. back to back to allow for an over-run in the web plate without interfering with the cover plates.

The radius of gyration about the axis A–A, Fig. 7, is approximately $0.45 \times 12.5 = 5.62$ in. (Table 136), and about the axis B–B is $0.23 \times 14 = 3.22''$ (Table 136). The axis B–B will control the design. The allowable unit stress is

$$f_c = 16{,}000 - 70\, l/r \text{ lb. per sq. in.} = 16{,}000 - 70 \times \frac{16 \times 12}{3.22} = 11{,}800 \text{ lb. per sq. in.}$$

which requires an area of

$$A = \frac{P}{f_c} = \frac{340{,}000}{11{,}800} = 28.8 \text{ sq. in.}$$

Try a section consisting of four angles 6″ × 4″ × ⅜″ with long legs turned out, and 12½ in. back to back, one web plate 12 in. × ⅜ in. and two flange plates 14 in. × ⅜ in. The properties of various sections are given in Table 70. The properties of sections are calculated as shown at the bottom of the table. The radius of gyration about the axis A–A is found to be $r_A = 5.58$ in., about the axis B–B is $r_B = 3.14$ in., and the area 29.44 sq. in.

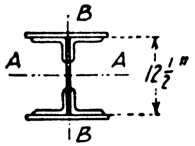


FIG. 7. PLATE AND ANGLE COLUMN.

For this section the ratio $l/r = 16 \times 12/3.14 = 61.2$ which satisfies the specification that the maximum value of l/r is 125. The allowable unit stress is,

$$f_c = 16{,}000 - 70 \times 61.2 = 11{,}700 \text{ lb. per sq. in.,}$$

and the required area is,

$$A = \frac{P}{f_c} = \frac{340{,}000}{11{,}700} = 29.1 \text{ sq. in.}$$

The area provided by the above section is 29.44 sq. in.

Expansion Rollers.—Design the rollers for the expansion end of a single track railway bridge of 175 ft. span, the dead load stress being 110,000 lb., the live load stress being 282,000 lb., and the impact 178,000 lb. Total stress = 570,000 lb.

References.—§ 7, p. 73; § 41, p. 189; § 81 to § 86, p. 192; § 19, p. 257; § 62, p. 260; § 38, p. 284; § 91, p. 287.

Solution.—The span being short a 6 in. roller will be used. The allowable stress per linear inch of rollers is $600 \times d$, when impact is considered, giving $600 \times 6 = 3{,}600$ lb. for 6 in. rollers.

The number of linear inches required is, $570{,}000/3{,}600 = 158$ in.

Five rollers 32 in. long provide $5 \times 32 = 160$ linear inches and occupy a space about 32 inches square.

MEMBERS IN FLEXURE.—The design of structural members stressed in flexure will be shown by several examples.

I-Beam.—Select an I-Beam to carry a uniform load of 1000 lb. per linear foot, the span being 16′ 0″ and the ends simply supported.

References.—§ 37, p. 76; § 50, p. 78; § 5, p. 131; § 7, p. 132; § 39, p. 189; § 50, p. 190; § 55, p. 191; § 17, p. 257; § 29 and § 30, p. 258; § 38, p. 284; § 48, p. 285; § 116, p. 289. Properties of I-beams and channels, are given in Tables 7 to 13, and Bethlehem beams in Tables 151 to 160, inclusive.

Solution.—The bending moment is

$M = \frac{1}{8}w \cdot l^2 = \frac{1}{8} \times 1000 \times 16^2 = 32{,}000$ ft.-lb. $= 32{,}000 \times 12$ in.-lb. $= 384{,}000$ in.-lb.

From applied mechanics,

$$M = \frac{f \cdot I}{c} = f \cdot S.$$

The section modulus required is then,

$$S = \frac{I}{c} = \frac{M}{f} = \frac{384{,}000}{16{,}000} = 24.0 \text{ in.}^3$$

The section modulus of a 9 in. I @ 35 lb. is 24.7 in.^{3}, and of a 10 in. I @ 25.4 lb. is 24.4 in.^{3} (Table 7), either of which will carry the load, but the 10 in. I @ 25.4 lb. being lighter is the more economical, and being the minimum section is more easily obtained.

The allowable bending moments in ft.-lb. for I-Beams, using a fiber stress of 16,000 lb. per sq. in., are given in Table 7. The I-Beam could have been selected directly from the moment making use of these values. The allowable bending moments for other unit stresses are proportional.

The safe uniform load, in tons, for I-Beams are given in Table 12, using a fiber stress of 16,000 lb. per sq. in. The I-Beam could have been selected directly from the load by using this table. Safe loads for other unit stresses are proportional.

If the I-Beam is not supported to prevent lateral deflection the allowable fiber stress must be reduced by the compression formula as shown in Table 12*a*.

Design an I-Beam 14′ 0″ long to carry a concentrated load of $P = 20{,}000$ lb. at the center of the beam. The maximum moment is at the center, and is, $M = \frac{1}{4}P \cdot l = \frac{1}{4} \times 20{,}000 \times 14 = 70{,}000$ ft.-lb. $= 840{,}000$ in.-lb.

The required section modulus is, $S = M/f = 840{,}000 \div 16{,}000 = 52.5$. In Table 7, the lightest beam that will carry the load is a 15 in. I @ 42.9 lb., which has a value of $S = 58.9$ in.^{3}, and a bending moment of 79,000 ft.-lb. A 12 in. I @ 55 lb. will also carry the load, but is not an economical section. A concentrated load, P, at the center will give the same maximum stresses as a uniformly distributed load of $2P$. From Table 12, a 15 in. I @ 42.9 lb. will carry a uniformly distributed load of 22 tons, which is sufficient.

Two I-Beams with Separators.—Design a girder consisting of two I-Beams fastened together by means of separators, the girder having a span of 16′ 0″ and carrying a uniform load of 2,000 lb. per linear ft.

Solution.—The bending moment is

$$M = \tfrac{1}{8}\, w.l^2 = \tfrac{1}{8} \times 2000 \times 16^2 = 64{,}000 \text{ ft.-lb.} = 798{,}000 \text{ in.-lb.}$$

From mechanics,

$$M = \frac{f \cdot I}{c} = f \cdot S.$$

The section modulus required is,

$$S = \frac{I}{c} = \frac{M}{f} = \frac{798{,}000}{16{,}000} = 48.0 \text{ in.}^3$$

Each I-Beam must have a section modulus of $\frac{1}{2} \times 48.0 = 24.0$ in.^{3} The section modulus of one 9 in. I @ 35 lb., is 24.7 in.^{3} and of one 10 in. I @ 25.4 lb., is 24.4 in.^{3}, either of which will carry one-half the load, but the 10 in. I @ 25.4 lb. being lighter is the more economical, and being the minimum section is more easily obtained.

The allowable bending moments, in ft.-lb. for I-Beams, using a fiber stress of 16,000 lbs. per

sq. in. are given in Table 7. The I-Beams could have been selected directly from the moment making use of these values.

The safe uniform load, in tons, for I-Beams is given in Table 12, using a fiber stress of 16,000 lb. per sq. in. The I-Beams could have been selected directly from the load using this table.

If the girder is not supported to prevent lateral deflection the allowable fiber stress must be reduced by the compression formula as shown in Table 12*a*.

The separators for Carnegie I-Beams are given in Fig. 4, page 107, Chap. II. The separators for Bethlehem beams are given in Table 158.

Plate Girders.—The full discussion of the design of plate girders would require more space than is available. The following notes will be of value.

References.—The following references should be consulted:

Weights.—Pages 146, 198 to 206.

Bending Moments and Shears.—Pages 207, 211, 212, 213, 214, 215, 216 to 221.

Unit Stresses.—§ 37, p. 76; § 47, p. 77; § 50, p. 78; § 51, p. 78; § 52, p. 78; § 5, p. 131; § 7, p. 132; § 37 to § 44, p. 189; § 50 to § 52, p. 190; § 15 to § 19, p. 257; § 29 to § 31, p. 258; § 77 to § 79, p. 260, § 38, p. 284; § 48, p. 285, § 116 to § 130, p. 289.

Proportions of Parts.—§ 3, p. 73; § 50 to § 54, p. 78; § 7, p. 132; § 3, p. 185; § 51, p. 190; p. 251; § 77, § 78, § 79, p. 260; § 80, p. 261; p. 267 to p. 272; § 51, p. 285; § 115, p. 288 to § 133, p. 290.

Details.—Pages 70, 168, 169, 237, 238.

The gross and net areas of angles are given in Table 29; Area of Plates, Table 1; Areas to be Deducted for Rivet Holes, Table 116; Moments of Inertia of Angles, Tables 32, 33 and 34; Moments of Inertia of Web Plates, Table 3; Moments of Inertia of Cover Plates, Table 5; Properties of Plate Girders, Table 87; Centers of Gravity of Plate Girder Flanges, Table 88.

Nomenclature.—The following nomenclature will be used.

M = resisting moment of section.
V = vertical shear at section.
f = allowable unit fiber stress.
I = moment of inertia of gross section.
I' = moment of inertia of net section.
I_w = moment of inertia of gross section of web plate.
I_w' = moment of inertia of net section of web plate.
A_F = gross area of one flange.
A_F' = net area of tension flange.
A_w = gross area of web.
h = distance between centers of gravity of flanges.
h' = distance between gage lines of rivets in tension and compression flanges.
d = distance back to back of angles in flanges.
c = distance from neutral axis to extreme fiber.
p = pitch of rivets in flanges.
r = allowable resistance of one rivet.
w = concentrated load per unit length of rail = P/l where P = concentrated load and l = distance over which the load, P, is considered as distributed (see § 5, p. 250).
$2n$ = number of rivets on one side of web splice.

Resisting Moment.—There are four methods now in use for determining the resisting moment of a plate girder section.

(1) Assuming that all the bending moment is carried by the flanges (see § 29, p. 254),

$$M = A_F' \cdot f \cdot h \qquad (1)$$

(2) Assuming that one-eighth the gross area of the web is available as flange area (see § 50, p. 78; § 50, p. 190; § 29, p. 258; § 116, p. 289),

$$M = (A_F' + \tfrac{1}{8}A_w) \cdot f \cdot h \qquad (1')$$

(3) By moment of inertia of net section,

$$M = \frac{f \cdot I'}{c} \qquad (1'')$$

(4) By moment of inertia of gross section (used by American Bridge Co. for plate girders for buildings),

$$M = \frac{f \cdot I}{c} \qquad (1''')$$

Rivets in Flanges Which do not Carry Concentrated Loads.

(1) Assuming that all bending moment is carried by flanges,

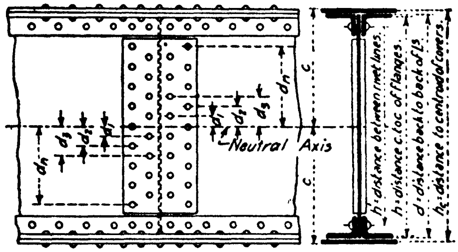


FIG. 8. WEB SPLICE FOR PLATE GIRDER.

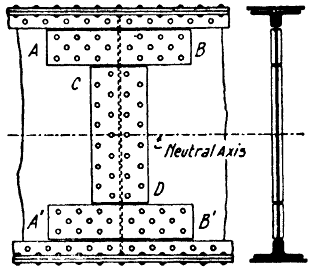


FIG. 9. WEB SPLICE FOR PLATE GIRDER.

$$p = \frac{r \cdot h'}{V} \qquad (2)$$

(2) Assuming that one-eighth the gross area of web is available as flange area,

$$p = \frac{A_{F}' + \frac{1}{8}A_w}{A_{F}'} \times \frac{r \cdot h'}{V} \qquad (3)$$

(3) By moment of inertia of net section,

$$p = \frac{2r \cdot I'}{V \cdot A_{F}' \cdot h} \qquad (4)$$

(4) By moment of inertia of gross section,

$$p = \frac{2r \cdot I}{V \cdot A_F \cdot h} \qquad (5)$$

Rivets in Flanges Carrying Concentrated Loads.

(1) Assuming that all the bending moment is carried by the flanges,

$$p = \frac{r}{\sqrt{w^2 + \left(\frac{V}{h'}\right)^2}} \qquad (6)$$

(2) Assuming that one-eighth the gross area of the web is available as flange area,

$$p = \frac{r}{\sqrt{w^2 + \left(\frac{A_{F}'}{A_{F}' + \frac{1}{8}A_w} \cdot \frac{V}{h}\right)^2}} \qquad (7)$$

(3) By moment of inertia of net section,

$$p = \frac{r}{\sqrt{w^2 + \left(\frac{V \cdot A_{F}' \cdot h}{2I'}\right)^2}} \qquad (8)$$

(4) By moment of inertia of gross section,

$$p = \frac{r}{\sqrt{w^2 + \left(\frac{V \cdot A_F \cdot h}{2I}\right)^2}} \tag{9}$$

Rivets Connecting Cover Plates to Flange Angles.

(1) and (2). Assuming that all the bending moment is carried by the flanges, or that one-eighth the gross area of the web is available as flange area,

$$p = \frac{n \cdot r \cdot d \cdot A_F}{V \cdot A_c'} \tag{10}$$

where n = number of rivets on one transverse line.
r = value of one rivet in single shear or bearing.
d = distance back to back of angles.
A_c' = total net area of cover plates in one flange.

(3) By moment of inertia of net section,

$$p = \frac{2n \cdot I' \cdot r}{V \cdot A_c' \cdot h_c} \tag{11}$$

where A_c' = total net area of cover plates in one flange.
h_c = distance between centroids of all cover plates in tension flange and all cover plates in compression flange.

(4) By moment of inertia of gross section,

$$p = \frac{2n \cdot I \cdot r}{V \cdot A_c \cdot h_c} \tag{12}$$

where A_c = total gross area of cover plates in one flange.
h_c = distance between centroids of all cover plates in tension flange and all cover plates in compression flange.

Web Splice.—An ordinary web splice is shown in Fig. 8. Where splice plates are designed to carry part of the moment as well as the shear the splice shown in Fig. 9 is sometimes used. Plates AB and $A'B'$ are assumed to transfer that part of the moment carried by the web, and plate CD to transfer the shear. Two lines of rivets should be used in each section of the web spliced. The number and spacing of rivets in a web splice can be determined only by trial, except when the first method for proportioning the section is used. The rivet most remote from the neutral axis is the most severely stressed.

(1) Assuming that all the bending moment is carried by the flanges,

$$r = \frac{V}{2n}, \text{ and } 2n = \frac{V}{r} \tag{13}$$

(2) Assuming that one-eighth the area of web is available as flange area. The stress in the outermost rivet is given by the formula, where M' is moment carried by web,

$$r = \sqrt{\left(\frac{V}{2n}\right)^2 + \left(\frac{M' \cdot d_n}{2\Sigma d^2}\right)^2} \tag{14}$$

(3) By moment of inertia of net section. The stress in the outermost rivet is given by the formula;

$$r = \sqrt{\left(\frac{V}{2n}\right)^2 + \left(\frac{I_w'}{I'} \cdot \frac{M \cdot d_n}{2\Sigma d^2}\right)^2} \tag{15}$$

(4) By moment of inertia of gross section. The stress in the outermost rivet is given by the formula

$$r = \sqrt{\left(\frac{V}{2n}\right)^2 + \left(\frac{I_w}{I} \cdot \frac{M \cdot d_n}{2\Sigma d^2}\right)^2} \tag{16}$$

For the details of a web splice, see Fig. 16.

Flange Splice.—Flanges should never be spliced unless it is impossible to get material of the required length. Flange splices should always be located at points where there is an excess of flange section, no two parts of the flange should be spliced within two feet of each other. Rivets in splice plates and angles should be located as close together as possible in order that the transfer may take place in a short distance. No allowance should be made for abutting edges of spliced members of the compression flange.

Flange angles should be spliced with a splice angle of equal section riveted to both legs of the angle spliced. Where this is impossible the largest possible splice angle should be used and the difference made up by a plate riveted to the vertical leg of the opposite angle. The number of rivets required in the splice angle on each side of the joint in the angle is given by the formula,

$$n = \frac{f \cdot A}{r} \tag{17}$$

where f = the allowable unit stress in the flange, A = area of spliced angle, and r = the allowable stress on one rivet. Rivets which are already considered as transferring the shear may be considered as splice rivets if they are included in the splice angle.

Cover plates should be spliced with a splice plate of equal section. The number of rivets required in the splice plate on each side of the joint is determined by the above formula if the plates are in direct contact in the same way as for splice angles. Where one or more plates intervene between the splice plate and cover plate which it splices, rivets should be used on each side of the joint in excess of the number required in case of direct contact, to an extent of one-third that number for each intervening plate.

The above methods for flange splicing apply only when methods (1) and (2) of proportioning sections are used, but may be used with sufficient accuracy when methods (3) and (4) are used. Strictly speaking for methods (3) and (4) splice angles and plates should have moments of inertia about the neutral axis, equal to the moments of inertia of the members they splice, about the neutral axis. An exact analysis for the number of rivets required in splices would give a less number than obtained from above formula.

Stiffeners.—For method of designing stiffeners see § 52, p. 78; § 7, p. 132; § 51, p. 190; § 79, p. 260; § 124, § 125, p. 289.

Pins and Pin Packing.—A pin under ordinary conditions is a short beam and must be designed (1) for bending, (2) for shear, and (3) for bearing. If a pin becomes bent the distribution of the loads and the calculation of the stresses are very uncertain.

The cross-bending stress, f, is found by means of the fundamental formula for flexure, $f = M \cdot c/I$, where the maximum bending moment, M, is found as explained later; I is the moment of inertia; and c is one-half the radius of a solid or hollow pin.

The safe shearing stresses given in standard specifications are for a uniform distribution of the shear over the entire cross-section, and the actual unit shearing stress to be used in designing will be equal to the maximum shear divided by the area of the cross-section of the pin.

The bearing stress is found by dividing the stress in the member by the bearing area of the pin, found by multiplying the thickness of the bearing plates by the diameter of the pin.

References.—§ 38, § 39, p. 76; § 40, p. 77; § 111, p. 84; § 39, § 40, § 41, p. 189; § 74, § 75, § 76, p. 192; § 92, p. 193; § 17, § 18, § 19, p. 257; §28, p. 258; § 52, p. 259; § 54, p. 259; § 136 p. 264; p. 267 to p. 268; § 38, p. 284; § 79, § 83, p. 287; p. 506.

Details of Pins.—Details of bridge pins are given in Table 95, Part II.

Stresses in Pins.—The method of calculation will be illustrated by calculating the stresses in the pin at U_1 in (*a*) Fig. 10. In the complete investigation of the pin U_1, it would be necessary to calculate the stresses when the stress in U_1U_2 was a maximum, and when the stress in U_1L_2 was a maximum. Only the case where the stress in U_1U_2 is a maximum will be considered. However, maximum stresses in pins sometimes occur when the stress in U_1L_2 is a maximum, and this case should be considered in practice.

Bending Moment.—The stresses in the members are shown in (*c*) Fig. 10, which gives the force polygon for the forces. The make-up of the members is shown in (*a*), and the pin packing on one side is shown in (*b*). The stresses shown in (*c*) are applied one-half on each side of the member, the pin acting like a simple beam. The stresses are assumed as applied at the centers of the plates which make the members.

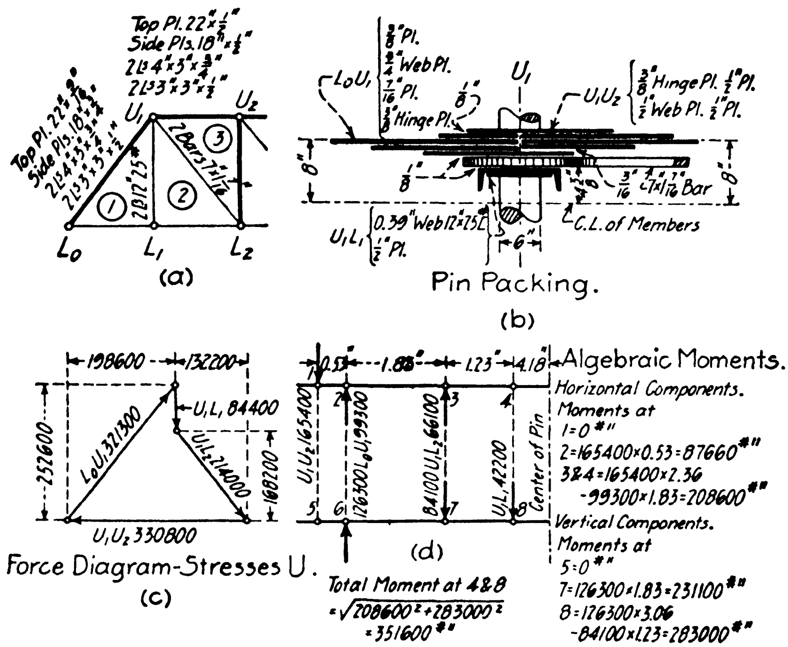


FIG. 10. CALCULATION OF STRESSES IN A PIN.

Calculation of Stresses in a Pin.—The amounts of the forces and the distances between their points of application as calculated from (*b*) are shown in (*d*) Fig. 10. The horizontal and vertical components of the forces are considered separately, the maximum horizontal bending moment and the maximum vertical bending moment are calculated for the same point, and the resultant moment is then found by means of the force triangle.

In (*d*) the horizontal bending moments are calculated about the points 1, 2, 3, 4; the maximum horizontal moment is to the right of 3, and is 208,600 in.-lb. The vertical bending moments are calculated about points 5, 6, 7, 8; the maximum bending moment is to the right of 8, and is 283,000 in.-lb. The maximum bending moment is at, and to the right of 4 and 8, and is, $M = \sqrt{208{,}600^2 + 283{,}000^2} = 351{,}600$ in.-lb. Substituting in the formula, $f = M \cdot c/I$, the maximum bending stress is $f = 16{,}600$ lb. per sq. in. The allowable bending stress in pins for which this bridge was designed was 18,000 lb. per square inch. The allowable bending moments on pin are given in Table 98.

Shear.—The shear is found for both the horizontal and vertical components as in a simple beam, and is equal to the summation of all the forces to the left of the section. The maximum horizontal shear is between 1 and 2, and is 165,400 lb. The shear between 2 and 3 is 165,400 − 99,300 = 66,100 lb. The maximum vertical shear is between 6 and 7, and is 126,300 lb. The resultant shear between 2 and 3, and 6 and 7, is, $V = \sqrt{126{,}300^2 + 66{,}100^2} = 145{,}000$ lb., which is less than the horizontal shear between 1 and 2. The maximum shear, therefore, comes

between 1 and 2, and is 165,400 lb. The maximum shearing unit stress is 165,400 ÷ 28.27 = 5,850 lb. per sq. in. The allowable shearing stress was 9,000 lb. per sq. in.

Bearing.—The bearing stress in L_0U_1 is 160,650 ÷ (6 × 1.94) = 13,800 lb. Bearing stress in U_1U_2 is 165,400 ÷ (6 × 1.88) = 14,600 lb. Bearing stress in U_1L_1 is 42,200 ÷ (6 × 0.89) = 7,900 lb. Bearing stress in U_1L_2 is 107,000 ÷ (6 × $1\frac{7}{16}$) = 12,400 lb. per sq. in. The allowable bearing stress was 15,000 lb. per sq. in. Allowable bearing stresses on pins are given in Table 97.

For the calculation of the stresses in pins, see the author's "Design of Highway Bridges of Steel, Timber and Concrete."

Pin Packing.—For details of pin packing see pages 267 and 268. Details of pins are given in Table 95, Part II.

Corrugated Steel Roofing.—For the calculation of the strength of corrugated steel and for a diagram for the safe loads for corrugated steel, see Fig. 18, Chap. I, page 22.

Bearing Plates.—The bearing plates required for beams and columns, Fig. 11, may be determined by the following formulas.

Let R = reaction of beam or load on column.
A = area of bearing plate.
w = allowable unit pressure in masonry.
f = allowable fiber stress in plate.
p = projection of bearing plate beyond any edge of beam or column.

Area of bearing plate,

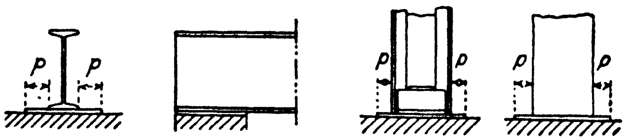


FIG. 11. BEARING PLATES.

$$A = \frac{R}{w} \tag{18}$$

Thickness of bearing plate required by a given projection,

$$t = p\sqrt{\frac{3R}{A \cdot f}} = p\sqrt{\frac{3w}{f}} \tag{19}$$

Safe projection for a given thickness of plate,

$$p = t\sqrt{\frac{A \cdot f}{3R}} = t\sqrt{\frac{f}{3w}} \tag{20}$$

The allowable pressures of bearing plates on masonry (value of w) are given in Table VIII, page 99. Standard bearing plates for I-beams are given in Table 8; for channels in Table 15. The length of I-beams which should bear on plates in order that the full shearing strength be developed is given in Table 11; and of channels in Table 16.

For a full discussion of bearing plates, see Bulletin No. 35, University of Illinois Engineering Experiment Station, entitled "A Study of Base and Bearing Plates for Columns and Beams," by Professor N. Clifford Ricker.

COMBINED FLEXURE AND DIRECT STRESS.—The formulas for combined flexure and direct stress are given in section 26, Chapter XVI. The design of members stressed in combined flexure and direct stress will be shown by several examples.

Eye-Bar.—An eye-bar in a structure carries a direct stress due to the dead and live loads, and in addition is stressed in flexure due to its own weight.

If P = direct stress in eye-bar; M_1 = bending moment due to weight in in.-lb.; c = distance from neutral axis to extreme fiber = $h/2$, where h = depth of eye-bar; l = length of bar, c. to c. of pins, t = thickness of eye-bar in inches; I = moment of inertia of eye-bar = $\frac{1}{12}\,t\cdot h^3$; k is a coefficient depending upon the condition of the ends being approximately 10 for eye-bars with pin ends, 24 for one pin end and one fixed end, and 32 for two fixed ends; E = modulus of elasticity of steel = 28,000,000 lb. per sq. in.; and $f_2 = \dfrac{P}{t\cdot h}$ = unit stress due to direct loads. Then the stress due to combined flexure and direct stress will be

$$f = f_2 + f_1 = \frac{P}{t\cdot h} + \frac{M_1\cdot c}{I + \dfrac{P\cdot l^2}{k\cdot E}} \tag{21}$$

Now, $M_1 = \frac{1}{8}w\cdot l^2$, where w = 0.28 $t\cdot h$ = the weight of the bar per lineal inch; $P = f_2\cdot t\cdot h$; $c = h/2$; $I = \frac{1}{12}t\cdot h^3$; k = 10; and E = 28,000,000 lb. per sq. in.; and substituting

$$f_1 = \frac{\frac{1}{8}w\cdot l^2\cdot\frac{1}{2}h}{\dfrac{b\cdot h^3}{12} + \dfrac{f_2\cdot b\cdot h\cdot l^2}{10\times 28{,}000{,}000}} = \frac{4{,}900{,}000h}{f_2 + 23{,}000{,}000\left(\dfrac{h}{l}\right)^2} \tag{22}$$

then f_1 is the extreme fiber stress in the bar due to weight, and is tension in the lower fiber and compression in the upper fiber.

If the bar is inclined, the stress obtained by formula (22) must be multiplied by the sine of the angle that the bar makes with a vertical line.

Diagram for Stress in Bars Due to Weight.—Taking the reciprocal of equation (22)

$$\frac{1}{f_1} = \frac{f_2}{4{,}900{,}000h} + \frac{23{,}000{,}000\left(\dfrac{h}{l}\right)^2}{4{,}900{,}000h} = y_1 + y_2$$

and

$$f_1 = \frac{1}{y_1 + y_2} \tag{23}$$

A diagram for solving equation (23) is given in Table 134, Part II, which see. The intersections of the inclined lines in Table 134 correspond to depths of eye-bar that give maximum stresses due to weight.

End-Post.—Design the end-post, Fig. 12, for a 160 ft. span through highway bridge. Panel length, 20′ 0″; depth of truss c. to c. of pins, 24′ 0″; length of end-post, 31′ 3″. The direct stresses are as follows: dead load stress = 30,000 lb.; live load stress = 60,000 lb.; impact = 100/(160 + 300) × 60,000 = 13,000 lb.; total direct stress due to dead load, live load and impact = 103,000 lb. The bridge is to be a class D_2 bridge designed according to the "General Specifications for Highway Bridges," in Chapter III. From § 38 of the specifications the allowable unit stress is f_c = 16,000 − $70l/r$. The section will be made of two channels and one cover plate. Try a section made of two 10 in. channels @ 15.3 lb., and one 14 in. by 5/16 in. plate, (*b*), Fig. 12. From Table 82, Part II, the radius of gyration about the horizontal axis A–A, is r_A = 3.99 in., and about the vertical axis B–B is, r_B = 4.67 in., and the eccentricity is, e = 1.70 in. The allowable stress is then $f_c = 16{,}000 - \dfrac{70\times 375}{3.99}$ = 9,400 lb. per sq. in. The required area will be = 103,000 ÷ 9,400 = 10.96 sq. in. The actual area is 13.30 sq. in. While the section appears to be excessive, it will be investigated for stress due to weight, eccentric loading and wind before rejecting it.

The area, radii of gyration and the eccentricity may be calculated as follows.

To calculate the area

area of two 10 in. channels (Table 14)	= 8.92 sq. in.
area of one 14 in. by 5/16 in. plate (Table 2)	= 4.38 sq. in.
Total area	= 13.30 sq. in.

To locate the neutral axis A–A, take moments about the lower edge of the channels

$$c = \frac{8.92 \times 5 + 4.38 \times 10.156}{13.30} = 6.70 \text{ in.}$$

The eccentricity is $e = 6.70 - 5.00 = 1.70$ in. The moment of inertia I_A, about axis A–A may be calculated as follows:

Let $I_c = I$ of channels about center of channels (Table 14).

$I_p = I$ of plate about center of plate (Table 4).

$A_c =$ area of channels (Table 14).

$A_p =$ area of plate (Table 1).

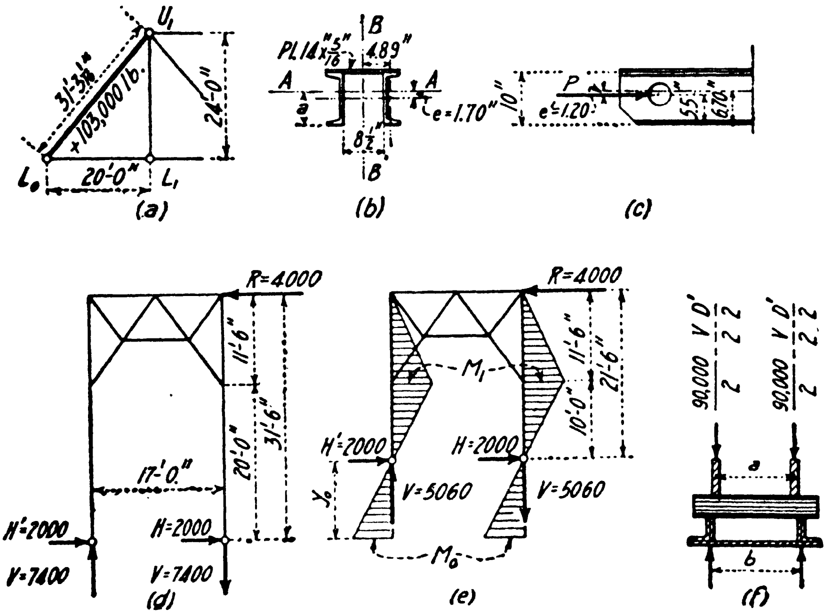


FIG. 12. END-POST OF A HIGHWAY BRIDGE.

Then

$$\begin{aligned} I_A &= I_c + I_p + A_c \times 1.70^2 + A_p \times 3.456^2. \\ &= 2 \times 66.9 + 0.04 + 8.92 \times 1.70^2 + 4.38 \times 3.456^2 \\ &= 133.8 + 0.04 + 25.76 + 52.20 \\ &= 211.80 \text{ in.}^4 \end{aligned}$$

Then $r_A = \sqrt{I_A \div A} = \sqrt{211.80 \div 13.3} = 3.99$ in.

The moment of inertia I_B, about axis B–B may be calculated as follows.

Let $I_c' = I$ of channels about neutral axis parallel to the web (Table 14).

$I_p' = I$ of plate about vertical axis (Table 3).

$A_c =$ area of channels (Table 14).

From Table 82 the distance back to back of channels is $8\frac{1}{2}$ in. From Table 14 the distance from neutral axis to back of channel is 0.639 in. The distance from neutral axis of channels to axis B–B is $4.25 + 0.639 = 4.889$ in. (4.89 in. will be used).

Then

$$\begin{aligned} I_B &= I_c' + I_p' + A_c \times 4.89^2 \\ &= 4.60 + 71.46 + 9.82 \times 4.89^2 \\ &= 4.60 + 71.46 + 213.28 \\ &= 289.34 \text{ in.}^4 \end{aligned}$$

Then $r_B = \sqrt{I_B \div A} = \sqrt{289.34 \div 13.3} = 4.67$ in.

Stress Due to Weight of Member.—The total weight of the member will be

Two 10 in. channels@15.3 lb., 31′ 6″ long	= 945 lb.
One 14 in. × 5/16 in. plate @ 14.88 lb., 30′ 0″ long	= 447 lb.
Details and lacing about 25 per cent	= 308 lb.
Total Weight, W	= 1700 lb.

The bending moment due to weight of member is $M = \frac{1}{8}W \cdot l \cdot \sin\theta$.
Stress due to weight

$$f_w = \frac{M \cdot c}{I_A - \dfrac{P \cdot l^2}{10E}} = \frac{\frac{1}{8}W \cdot l \cdot \sin\theta \cdot x}{I_A - \dfrac{P \cdot l^2}{10E}} \tag{25}$$

The stress due to weight in the upper fiber will be

$$f_w = \frac{\frac{1}{8} \times 1{,}700 \times 375 \times 0.645 \times 3.6125}{211.8 - \dfrac{103{,}000 \times 375^2}{10 \times 30{,}000{,}000}}$$
$$= 940 \text{ lb. per sq. in.}$$

The stress due to weight in the lower fiber is

$$f'_w = -6.70 \times 940 \div 3.6125$$
$$= -1745 \text{ lb. per sq. in.}$$

Stress Due to Eccentric Loading.—The pins were placed ½ inch above the center of the channels, and the stress due to eccentric loading will be

$$f_e = \frac{M_1 \cdot c}{I - \dfrac{P \cdot l^2}{10E}} = \frac{P \times (1.70 - 0.5) \times c}{I - \dfrac{P \cdot l^2}{10E}} \tag{26}$$

The eccentric stress in the upper fiber will be

$$f_e = \frac{103{,}000 \times 1.20 \times 3.6125}{211.8 - \dfrac{103{,}000 \times 375^2}{10 \times 30{,}000{,}000}}$$
$$= -2{,}280 \text{ lb. per sq. in.}$$

The eccentric stress in the lower fiber is

$$f_e = +6.70 \times 2{,}280 \div 3.6125$$
$$= +4{,}230 \text{ lb. per sq. in.}$$

The resultant stress due to weight and eccentric loading is $f_1 = f_w + f_e = +940 - 2{,}280 = -1{,}340$ lb. in the upper fiber, and $-1{,}745 + 4{,}230 = 2{,}485$ lb. per sq. in. in the lower fiber.

The allowable stress due to weight and eccentric loading is greater than 10 per cent of the allowable stress and must be considered, with the allowable unit stress increased by 10 per cent (§ 48, p. 190).

The total unit stress in the member will be, $f = 103{,}000 \div 13.30 + 2{,}485 = 7{,}752 + 2{,}485 = 10{,}237$ lb. per sq. in. The allowable unit stress when weight and eccentric loading are considered is $9{,}400 \times 1.10 = 10{,}340$ lb. per sq. in., which is sufficient.

Stress Due to Wind Moment.—The stresses in the portal and the direct wind stresses in the end-post when the end-post is assumed as pin-connected at the base are shown in (*d*) and (*e*) Fig. 12. The end-posts may both be assumed as fixed if the windward end-post is fixed. To fix the windward end-post the bending moment must not be greater than the resisting moment which will be

$$M_o = H \cdot y_o = (90{,}000 - V - D')a/2$$

where $V = 5{,}060$ lb. and $D' = 7{,}000$ lb. the direct stress due to wind, and a = distance center to center of metal in the sides of the end-post = 8.87 in., (*f*), Fig. 12. (The impact stress is omitted.) If y_o is taken equal to $\frac{1}{2}d$ = 10′ 0″ = 120 in., we will have

$$2{,}000 \times 120 \leqq (90{,}000 - 5{,}060 - 7{,}000)\ 8.87/2$$

which makes 240,000 < 345,600, and the end-post may be assumed as fixed at the base.

The stress due to bending moment due to wind loads in the leeward end-post will be,

$$f_w = \frac{M \cdot c}{I - \dfrac{P \cdot l^2}{10E}} \tag{27}$$

$$= \frac{240{,}000 \times 7}{289.4 - \dfrac{(90{,}000 + 5{,}060 + 7{,}000)258^2}{10 \times 30{,}000{,}000}} = 6{,}730 \text{ lb. per sq. in.}$$

The total stress due to direct wind load will be $f_w = (5060 + 7000)/13.30 = +910$ lb. per sq. in. The total maximum wind load stress will come on the windward fiber of the leeward end-post, and will be $f_w'' = +6{,}370 + 910 = +7{,}280$ lb. per sq in.

The maximum stress due to direct dead and live loads (not including impact) and wind load stresses will be

$$f = 90{,}000 \div 13.30 + 7{,}280$$
$$= 6{,}770 + 7{,}280 = 14{,}050 \text{ lb. per sq. in.}$$

From § 46 in the specifications the allowable stress may be increased 50 per cent when direct and flexural wind stresses are considered.

The allowable stress when both direct and flexural wind stress are considered is then

$$f_c = 9{,}400 \times 1.50 = 14{,}000 \text{ lb. per sq. in.}$$

The stresses in the windward post will be less than in the leeward end-post calculated above.

While the section assumed appeared to be excessive, the additional area and the width of plate are required to take the flexure due to wind loads.

For the method used by the C. M. & St. P. Ry. for the design of an end-post, see p. 270.

Column of a Transverse Bent.—Design a column similar to that of the transverse bent shown in Fig. 3, Chapter XVI, but having column length of 25′ 6″ and being hinged at the base. Direct stress = + 12,800 lb., bending moment at foot of knee brace = 181,250 ft.-lb. Shear = H = 13,500 lb.

References.—§ 36, p. 76; § 41, § 44, p. 77; § 106, § 108, p. 83.

Solution.—A section composed of four angles and a plate will be used. The column will be supported laterally by the girts so the length in that direction will be taken as ½ × 25′ 6″ = 12.75 ft.

Try 4 angles 5″ × 3½″ × ½″, long legs out, 18½ in. back to back and one web plate 18 in. × ⅜ in. Distance between rivet lines = 18½ − 2 × 2 = 14½ in. Maximum allowable distance for ⅜ in. plate = 40 × ⅜ = 15 in.

Using method at bottom of Table 69, $A = 22.75$ in.²; $I_A = 1{,}311$ in.⁴; $I_B = 94.6$ in.⁴; $r_A = 7.59$ in.; $r_B = 2.04$ in. The greatest value of $l \div r = 12.75 \times 12 \div 2.04 = 75.0$. The maximum allowable value of $l \div r = 125$. The allowable unit stress is:

$$1.50(16{,}000 - 70\, l/r) = 1.50(16{,}000 - 70 \times 75.0) = 16{,}100 \text{ lb. per sq. in.}$$

The actual unit stress is:

$$S = \frac{P}{A} + \frac{M \cdot c}{I - \dfrac{P \cdot l^2}{10E}} = \frac{12{,}800}{22.75} + \frac{181{,}250 \times 12 \times 9.25}{1311 - \dfrac{12{,}800 \times 25.5^2 \times 12^2}{10 \times 30{,}000{,}000}} = 16{,}000 \text{ lb. per sq. in.}$$

Floorbeam.—Floorbeams are designed in the same way as other plate girders. The section cut away for clearance at the joint must be strengthened by means of plates as shown in Fig. 13. To determine the strength at the weakest section, A–A, the following method is used.

The floorbeam is drawn to scale in Fig. 13, so that distances can be scaled and the maximum floorbeam reaction 189,980 lb. be resolved graphically, in the center line of the post, into 80,000 lb. normal to A–A, which produces direct tension on the section A–A, and 173,000 lb. parallel to A–A, which produces shear and flexural stress.

Rivet holes are considered as spaced 3 in. along the section $A-A$, for when the beam is detailed it is not probable that they will be spaced closer than 3 in. Holes are deducted from the tension side only. 1 in. holes being deducted for ⅞ in. rivets.

The plates may not be exactly as indicated on Fig. 13 for it may be necessary to alter them slightly in detailing, but small changes will not change the results materially. It is quite an advantage to have the investigation made before the beam is completely detailed as alterations are more easily made at that time if the beam proves weak in any particular.

The curved angle at the bottom will not be considered as adding to the strength.

Values for the area, eccentricity and moment of inertia are found as follows.

First the moments and moments of inertia of the separate parts are found about an axis through the geometric center of the section, the eccentricity is then calculated. The moment of inertia about an axis through the center of gravity is found by subtracting the product of the

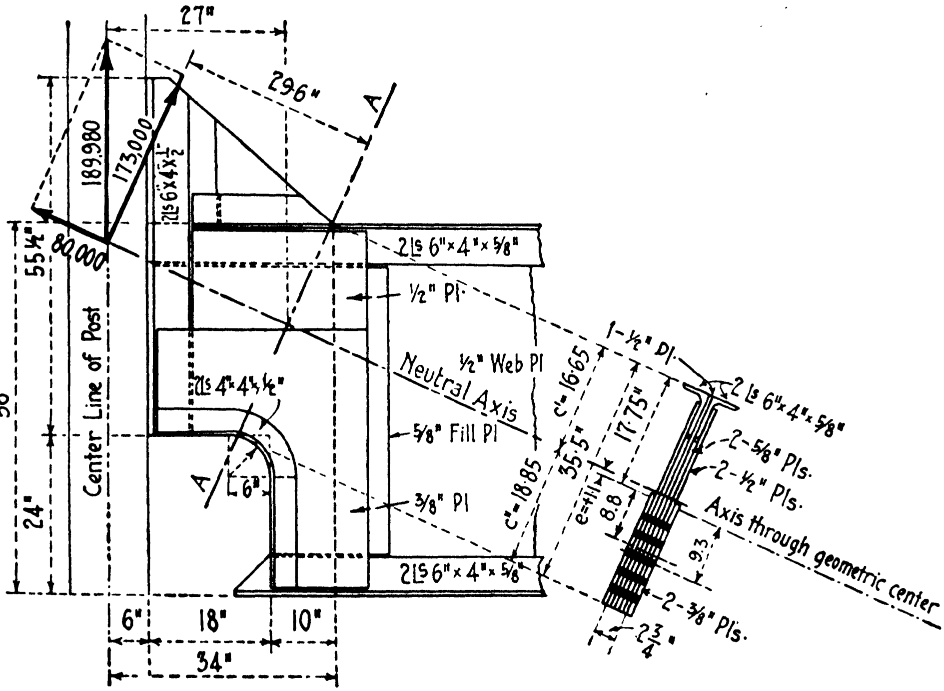


FIG. 13. DETAIL OF FLOORBEAM CONNECTION.

area and the eccentricity squared from the moment of inertia about the axis through the geometric center or

$$I_c = I_m - A \cdot e^2$$

Note.—For sake of simplicity the total section was divided up as follows:

A, includes three ½ in. and two ⅝ in. plates, the 6″ × ⅝″ legs of the flange angles and ⅝ in. +½ in. of the 4″ × ⅝″ leg. The spaces allowed for clearance were considered as solid with no appreciable error.

B, includes the remainder of the 4″ × ⅝″ legs of flange angles.

C, includes the ⅜ in. outside plates considered as solid.

D, includes the rivet holes, 1 in. in diameter and 3.5 in. long, spaced 3 in. center to center.

TABLES OF AREAS, MOMENTS AND MOMENTS OF INERTIA

Section.	Size, In.	Area, Sq. In.	Y_0, In.	Moment, In.-Lb.	Y_0, In.	I_m, In⁴.
A	35.5 × 2.75	+97.6	0	0	0	0
		Moment of Inertia about own axis				+10,250
B	5.75 × 0.625	+ 3.6	+17.4	+ 62.6	+17.4	+ 1,088
		Moment of Inertia about own axis				0
C	18.0 × 0.75	+13.5	− 8.8	−118.6	− 8.8	+ 1,044
		Moment of Inertia about own axis				+ 365
						12,747
D	5 × 1 × 3.5	−17.5	− 9.3	+162.6	− 9.3	− 1,513
		Moment of Inertia about own axis				− 315
		+97.2		+106.6		10,919
e = 106.6 ÷ 97.2 = 1.10		$A \cdot e^2$ = 97 2 × 1.10² =				− 117
Total moment of inertia about centroidal axis =						10,802

The bending moment of this section, from Fig 14 is

$$M = 189{,}980 \times 27 = 5{,}130{,}000 \text{ in.-lb.}$$

or

$$M = 173{,}000 \times 29.5 = 5{,}130{,}000 \text{ in.-lb}$$

The direct tension is 80,000 lb.

The shear on the section is 173,000 lb.

Compression in extreme fiber due to moment

$$S_1 = M \cdot c' \div I = (5{,}130{,}000 \times 16.65) \div 10{,}802 = +\ 7{,}850 \text{ lb. per sq. in.}$$

Tension in extreme fiber due to moment is

$$S_1 = M \cdot c''/I = 5{,}130{,}000 \times 18.85 \div 10{,}802 = -\ 8{,}950 \text{ lb. per sq. in.}$$

Tension on whole section due to direct stress

$$S_2 = P/a = 80{,}000 \div 97.2 = -\ 820 \text{ lb. per sq. in.}$$

Total compression in extreme fiber

$$S = S_1 + S_2 = 7{,}850 - 820 = +\ 7{,}030 \text{ lb. per sq. in.}$$

Total tension in extreme fiber

$$S = S_1 + S_2 = -\ 8{,}950 - 820 = -\ 9{,}770 \text{ lb. per sq. in.}$$

Unit shear is approximately

$$S = 173{,}000 \div 97.2 = 1{,}780 \text{ lb. per sq. in.}$$

The allowable unit stress in compression = 16,000 lb. per sq. in.

The allowable unit stress in tension = 16,000 lb. per sq. in.

The allowable unit stress in shear = 10,000 lb. per sq. in.

END CONNECTIONS FOR TENSION AND COMPRESSION MEMBERS.—For simple connections with concentric stresses the number of rivets in riveted end connections may be taken as equal to the total stress in the member divided by the allowable stress on one rivet for bearing or for shear, Table 114, whichever gives the larger number of rivets. Specifications uniformly require that the connections of members be designed to develop the full strength of the member. The minimum number of rivets in shop connections should be two rivets, except for lacing bars; while the minimum number of rivets in field connections should be three rivets, except for lacing bars. In lateral bracing or stiff bracing or struts the actual number of rivets required to develop the full strength of the member should be increased by two rivets, for the reason that two rivet holes are almost certain to be badly distorted by the drift pins in drawing the member up. Rivets should be grouped symmetrically about the neutral axis of the member or the eccentric stresses should be calculated and provided for. The strength of a structure depends very much upon the strength of the connections, and the details of the joints and connections should be worked out with great care.

References.—§ 92, § 96, § 98, § 109, p. 83; § 110, § 111, p. 84; § 13, § 14, § 15, p. 133; § 116, p. 134; § 40, § 41, p. 189; § 60, § 62, § 66, § 67, § 69, § 70, p. 191; § 37, § 39, § 43, p. 44; § 46, § 47, § 52, p. 259; § 57, § 60, p. 285; § 65, § 67, § 69 to § 73, p. 286; § 79, § 80, p. 287.

Strut or Tie.—Design the end connection for a 4" x 4" x 3/8" angle, carrying a stress (either tensile or compressive) of 40,000 lb., the angle being fastened by both legs to a 3/8 in. plate as shown in Fig. 2, using 3/4 in. rivets.

Solution.—The allowable stress on one 3/4 in. rivet in single shear is 5,300 lb. and in bearing on a 3/8 in. plate is 6,750 lb., using 12,000 lb. per sq. in. and 24,000 lb. per sq. in. as the allowable stresses in shear and bearing, respectively. Table 114. The shear evidently controls, and the number of rivets is

$$n = \frac{40,000}{5,300} = 7.6 \text{ or 8 rivets.}$$

Four of these will be placed in the main angle and four in the lug angle. In order to transfer the proper portion of the stress to the lug angle, the number of rivets between the main angle and lug angle must be equal to the number of rivets in the lug angle, or four in this case.

If the angle is connected by one leg only the eight rivets will be put in one leg as shown in Fig. 3.

Pin-connected Top Chord.—Design the end connection for the top chord of a pin-connected bridge as shown in Fig. 14. Length center to center of pins = 25′ 0″. Rivets 7/8 in.

Solution.—The connections should be designed to carry the full strength of the member and not the stress that it carries. The allowable unit stress is $f_c = 16,000 - 70\, l/r = 16,000 - 70 \times \frac{25 \times 12}{8.12} = 13,420$ lb. per sq. in. Total stress $= 13,420 \times 51.84 = 695,700$ lb.

The entire stress of 695,000 lb. must be transferred from the member to the pin through the pin plates and web plates. In the body of the member the stress is distributed among the different parts in proportion to the gross area, or as follows:

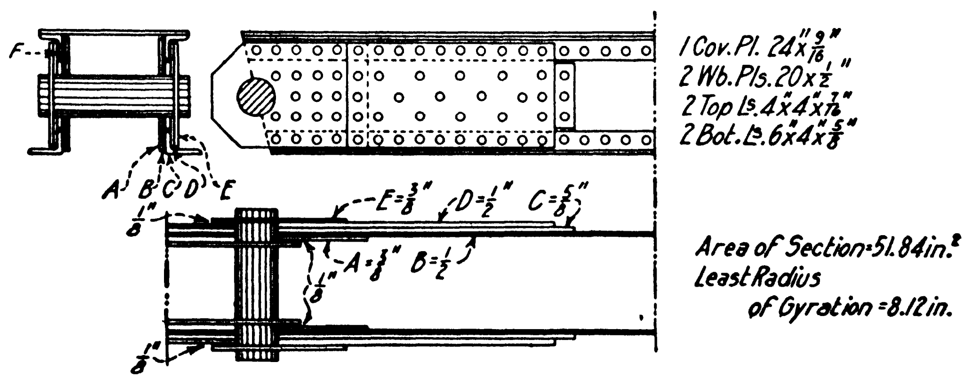


FIG. 14. END CONNECTION OF TOP CHORD.

Item.	Material.	Area × Unit Stress = Total Stress.	Stress on One Side.
1 Cover Plate	24 in. × 9/16 in.	13.50 × 13,420 = 181,000 lb.	90,500 lb.
2 Top Angles	4" × 4" × 7/16"	6.62 " = 88,900 "	44,450 "
2 Web Plates	20 in. × 1/2 in.	20.00 " = 268,500 "	134,250 "
2 Bottom Angles	6" × 4" × 5/8"	11.72 " = 157,300 "	78,650 "
		51.84 × 13,420 = 695,700 lb.	347,850 lb

The total bearing area required on one side of the member is,

$$A = \frac{347,850}{24,000} = 14.49 \text{ sq. in.}$$

The total thickness of bearing required on one side, using a 6¼ in. pin, is,

$$t = \frac{14.49}{6.25} = 2.32 \text{ in.}$$

This thickness will be provided by the plates A, B, C, D and E as shown in Fig. 14. The plate B in the web and has a thickness of ½ in. Plate C must act as a fill plate so must be of the same thickness as the bottom angles or ⅝ in. The outside plate E and the inside plate A should be thinner than D so they will be made ⅜ in., and D will be made ½ in. The actual thickness of bearing is then 2.375 in., and the required thickness is 2.32 in. In arranging the plates a clearance of ⅛ in. should be allowed between the plates which pass around the pin, and the nearest plate as shown in Fig. 14. It is necessary to put a 3/16 in. fill plate, F, opposite the top angle to make up for the difference in thickness in the ⅝ in. bottom angle and the 7/16 in. top angle.

The stress transmitted to a plate by the pin is equal to the ratio of its thickness to the total thickness, multiplied by the total stress. The stresses in the various plates are as follows.

$$\begin{aligned}
\text{Stress in } A &= \frac{0.375}{2.375} \times 347,850 = 54,920 \text{ lb.}\\
B &= \frac{0.500}{2.375} \times 347,850 = 73,240 \text{ lb.}\\
C &= \frac{0.625}{2.375} \times 347,850 = 91,530 \text{ lb.}\\
D &= \frac{0.500}{2.375} \times 347,850 = 73,240 \text{ lb.}\\
E &= \frac{0.375}{2.375} \times 347,850 = \underline{54,920} \text{ lb.}\\
\text{Total} &= 347,850 \text{ lb.}
\end{aligned}$$

An exact solution for the number and location of rivets is not practicable. A common solution is to consider that all the pin plates transmit their stress to the web and that the web, in turn, distributes this stress over the section. This solution overstresses the web in the vicinity of the pin.

A better solution is to consider that the stress in the cover plate and top angles is transmitted in double shear or bearing on the vertical leg of the top angles from the web plates and pin plates through the rivets in the vertical leg of the angles. The stress in the bottom angles is transmitted in double shear or bearing on the vertical leg of the bottom angles from the web plates and pin plates through the rivets in the vertical leg of the angles. The stress on the rivets between the web plate and plate C is equal to the sum of the stresses in C, D and E, minus one-half the sum of the stresses in the cover plate, top angles and bottom angles on one side.

The number of rivets in the plate A is determined by the stress in A only, and is controlled by single shear and is,

$$n = \frac{54,920}{7,220} = 8 \text{ rivets.}$$

The number of rivets in the plate E is determined by the stress in E only, and is controlled by single shear and is,

$$n = \frac{54,920}{7,220} = 8 \text{ rivets.}$$

The number of rivets between D and the top angle and between B and the top angle is determined by bearing on the 7/16 in. angle and is,

$$n = \frac{90,500 + 44,450}{9,190} = 15 \text{ rivets.}$$

The number of rivets between D and the bottom angle and between B and the bottom angle is,

$$n = \frac{78,650}{9,190} = 9 \text{ rivets.}$$

The number of rivets between C and web, B, is determined by single shear, and is

$$n = \frac{73,240 + 54,920 + 91,530 - \frac{1}{2}(90,500 + 44,450 + 78,650)}{7,220} = 16 \text{ rivets.}$$

End Connections for I-Beams.—The end connections for Carnegie I-Beams are given in Tables 117 and 118, and for Bethlehem I and Girder Beams in Tables 156 and 157, respectively. The end connections for short beams, and for beams carrying heavy loads should be carefully investigated for direct and bending stresses. Rivets should never be used in direct tension, Connections where rivets would be in direct tension should be provided with turned bolts.

Eccentric Riveted Connections.—The actual shearing stresses in riveted connections are often very much in excess of the direct shearing stresses. This will be illustrated by the calculation of the shearing stresses in the rivets in the standard connection shown in Fig. 15, which is assumed as loosely bolted to a column.

The eccentric force, P, may be replaced by a direct force, P, acting through the center of gravity of the rivets and parallel to its original direction, and a couple with a moment $M = P \times 3$ in. = 60,000 in.-lb. Each rivet in the connection will then take a direct shear equal to P divided by n, where n is the total number of rivets in the connection, and a shear due to bending moment M.

The shear in any rivet due to moment will vary as the distance, and the resisting moment exerted by each rivet will vary as the square of the distance of the rivet from the center of gravity of all the rivets.

Now, if a is taken as the resultant shear due to bending moment in a rivet at a unit's distance from the center of gravity, we will have the relation,

$$\begin{aligned} M &= a(d_1^2 + d_2^2 + d_3^2 + d_4^2 + d_5^2) \\ &= a\Sigma d^2 \end{aligned}$$

and

$$a = \frac{M}{\Sigma d^2} = \frac{60,000}{23.10} = 2,600 \text{ lb.} \qquad (27)$$

The remainder of the calculations are shown in Table I. The resultant shears on the rivets are given in the last column of the table and are much larger than would be expected.

The force and equilibrium polygons for the resultant shears and load P, drawn in Fig. 15, close, which shows that the connection is in equilibrium.

TABLE I.

Direct shear, $S = 20,000 \div 5 = 4,000$ lb
Moment $= 20,000 \times 3 = 60,000$ in.-lb. $= a(d_1^2 + d_2^2 + d_3^2 + d_4^2 + d_5^2)$ where a = moment shear on rivet 3 = 2,600 lb.

Rivet.	d, In.	d^2, In^2.	Moment, In.-Lb.	M, Lb.	S, Lb.	R, Lb.
1	2.70	7.25	18,850	6,820	4,000	9,260
2	1.95	3.80	9,875	5,070	4,000	3,250
3	1.00	1.00	2,600	2,600	4,000	6,600
4	1.95	3.80	9,875	5,070	4,000	3,250
5	2.70	7.25	18,850	6,820	4,000	9,260
		23.10	60,000		20,000	

$a\Sigma d^2 = 23.10\, a = 60,000$ in.-lb.
$a = 2,600$ lb. = moment shear on rivet 3
M = shear due to moment
S = shear due to direct load, P.
R = resultant shear

Center of Motion.—The total shear on rivet 3 is 4,000 + 2,600 = 6,600 lb., and is parallel to the resultant force P. There will be some point to the right of rivet 3 where the total shear on a rivet will be zero. This point will be at a distance to the right rivet 3 equal to 6,600 + 2,600 = 2.54 in. The center of motion of all of the rivets of the group will then be at a distance to the right of the center of gravity of the rivets equal to 2.54 − 1.00 = 1.54 in. The total shear on each rivet will be equal to a (2,600 lb.) multiplied by the distance of the rivet from the center of motion. The direction of the shear on each rivet will be normal to the rotation arm.

Let the distance from the center of motion to any rivet be represented by the distance s, and the distances of the rivets will be

$$s_3 = 2.54 \text{ in.}, \; s_2 = s_4 = \sqrt{1.25^2 + 0.04^2} = 1.25 \text{ in.}, \; s_1 = s_5 = \sqrt{2.54^2 + 2.5^2} = 3.56 \text{ in.}$$

The total shears on the rivets will then be

$$R_3 = 2.54 \times 2{,}600 = 6{,}600 \text{ lb.}$$
$$R_2 = R_4 = 1.25 \times 2{,}600 = 3{,}250 \text{ lb.}$$
$$R_1 = R_5 = 3.56 \times 2{,}600 = 9{,}260 \text{ lb.}$$

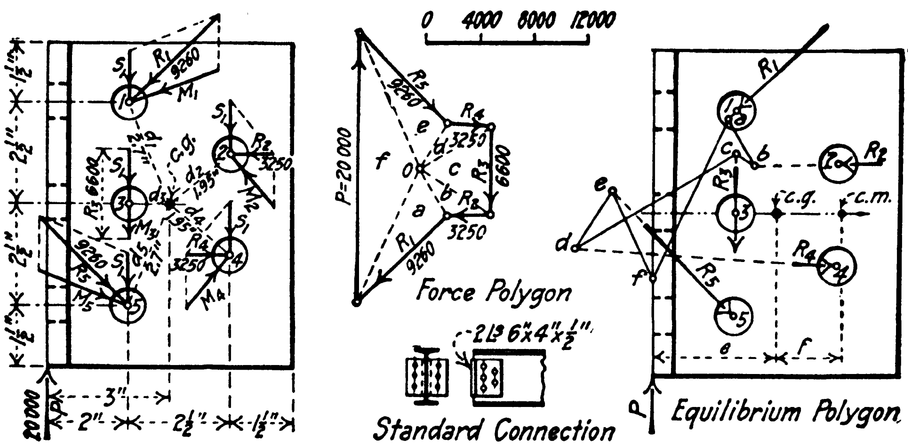


FIG. 15.

The value of Σd^2 about the center of gravity of the rivets may be calculated as follows. If the coordinates of the rivets are represented by x and y, then

$$\Sigma d^2 = \Sigma(x^2 + y^2) = 23.10 \text{ in.}^2$$

and $a = M/\Sigma d^2 = 2{,}600$ lb. as above. Let e be the distance from the line of action of P to the center of gravity of the rivets and f be the distance from the center of gravity to the center of motion, Fig. 15. Now the moment of the direct shears on the rivets about the line of action of force P will be equal to the twisting moment of the rivets about the center of gravity, and since the direct shears on the rivets parallel to line of action of force P is $f.n.a.$,

$$f \times n \times a \times e = a\Sigma d^2$$

and

$$f = \frac{\Sigma d^2}{n \cdot e} \tag{a}$$

and since number of rivets is $n = 5$

$$f = \frac{\Sigma d^2}{5e} = 23.10/15 = 1.54 \text{ in.}$$

which gives a method for calculating f for any connection. The center of motion will be on a line drawn normal to the line of action of P and passing through the center of gravity.

From (27) $\Sigma d^2 = M/a = P \cdot e/a$, and

$$f = P/(n \cdot a) \tag{27a}$$

Stress in Extreme Rivet of a Group.—Now if x_1 and y_1 are the coordinates of the rivet 1 with reference to the center of gravity of the rivets, and z_1 is the distance from center of motion to rivet number 1, then

$$z_1^2 = (x_1 + f)^2 + y_1^2$$

substituting value of $f = \Sigma d^2/n \cdot e$, and solving

$$z_1 = \sqrt{\left(x_1 + \frac{\Sigma d^2}{n \cdot e}\right)^2 + y_1^2}$$

Now the maximum stress on rivet number 1 will be $R_1 = a \cdot z_1$, and

$$R_1 = a\sqrt{\left(x_1 + \frac{\Sigma d^2}{n \cdot e}\right)^2 + y_1^2} \tag{27b}$$

also since $f = P/(n \cdot a)$

$$R_1 = \sqrt{(a \cdot x_1 + P/n)^2 + (a \cdot y_1)^2} \tag{27c}$$

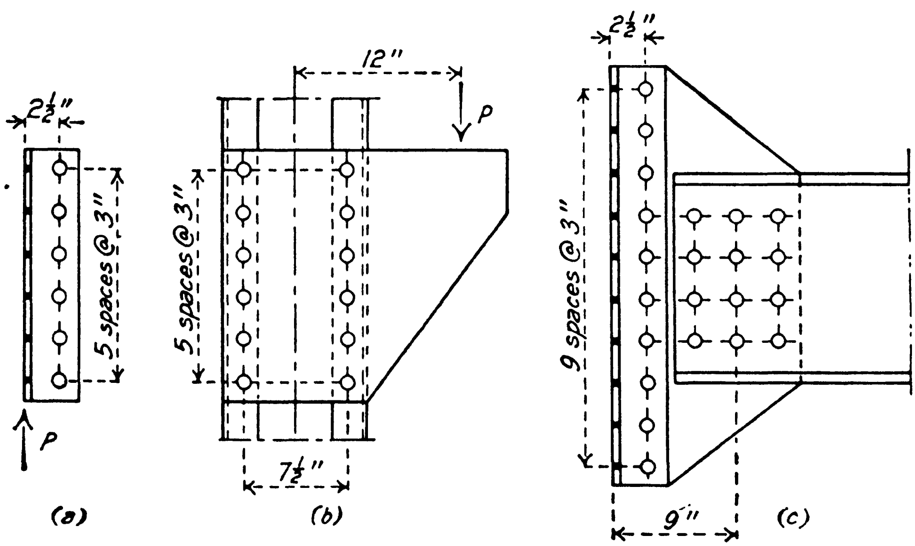


FIG. 15a.

Example 1.—Calculate the maximum shear on the rivets in the connection angle for a 24 in. I-beam in (a) Fig. 15a. There are 6 rivets spaced 3 in., $e = 2\frac{1}{2}$ in.

$$\Sigma d^2 = 2(1.5^2 + 4.5^2 + 7.5^2) = 157.5$$
$$a = P \cdot e/\Sigma d^2 = 2.5P/157.5 = P/63 \text{ lb.}$$
$$f = \Sigma d^2/n \cdot e = 157.5/(6 \times 2.5) = 10.5 \text{ in.}$$

For top rivet $x_1 = 0$, $y_1 = 7.5$ in.

From (27*b*)

$$R_1 = \frac{P}{63}\sqrt{10.5^2 + 7.5^2}$$
$$= .2P = P/5$$

Example 2.—Calculate maximum shear on right hand top rivet in connection plate in (*b*) Fig. 15*a*. $n = 12$. $e = 12$ in. Rivet spacing 3 in.

$$\Sigma d^2 = \Sigma x^2 + \Sigma y^2 = 12(3.75)^2 + 4(1.5^2 + 4.5^2 + 7.5^2)$$
$$= 484$$
$$a = P \cdot e/\Sigma d^2 = P \times 12/484 = P/40.3 \text{ lb.}$$
$$f = \Sigma d^2/(n \cdot e) = 484/12 \times 12 = 3.36 \text{ in.}$$

Now for top right hand rivet, $x_1 = 3.75$ in. and $y_1 = 7.5$ in.
From (27*b*)

$$R_1 = \frac{P}{40.3}\sqrt{(3.75 + 3.36)^2 + 7.5^2}$$
$$= P/4$$

This is the problem solved at the bottom of Table 118*b*, Part II.

For $\frac{3}{4}$-in. rivets, with a unit stress of 12,000 lb. per sq. in. in shear, the safe load on the bracket will be

$$P = 5.300 \times 4 = 21,200 \text{ lb.}$$

Example 3.—Calculate the tensile stress due to a moment $M = 250,000$ in.-lb. in the upper rivet of the outstanding leg of the connection angle in (*c*) Fig. 15*a*. $n = 10$. $y_1 = 13.5$ in. $x_1 = 0$. Rivets $\frac{3}{4}$ in. Rivets spaced 3 in.

$$\Sigma d^2 = 2(1.5^2 + 4.5^2 + 7.5^2 + 10.5^2 + 13.5^2)$$
$$= 742$$
$$a = M/\Sigma d^2 = 250,000/742 = 337 \text{ lb.}$$
$$f = 0, \quad \text{also} \quad x_1 = 0$$

and since $s_1 = y_1 = 13.5$ in.,

$$R_1 = 13.5a = 13.5 \times 337 = 4,550 \text{ lb.}$$

For $\frac{3}{4}$-in. rivets with 12,000 lb. per sq. in. shear the allowable shear = 5.300 lb. If tensile stress is taken equal to shear this is safe.

The direct shear and tensile stress on the rivet are independent of each other and need not be combined.

Example 4.—Calculate the stresses in the plate connecting to the channel in (*c*) Fig. 15*a*, due to a vertical load of $P = 18,000$ lb. Rivets $\frac{3}{4}$ in. $n = 12$. $e = 9$ in. Rivets spaced 3 in.

$$\Sigma d^2 = \Sigma x^2 + \Sigma y^2 = 8 \times 3^2 + 6(1.5^2 + 4.5^2)$$
$$= 207$$
$$a = P \cdot e/\Sigma d^2 = 18,000 \times 9/207 = 783 \text{ lb.}$$
$$f = \Sigma d^2/(n \cdot e) = 207/(12 \times 12) = 1.44$$

from (27*b*)

$$R_1 = 783\sqrt{(3 + 1.44)^2 + 4.5^2}$$
$$= 5,030 \text{ lb.}$$

For a unit shear of 12,000 lb. per sq. in. the allowable shear on a $\frac{3}{4}$-in. rivet = 5,300 lb.

Web Splice.—The plate girder shown in Fig. 16 is to be spliced at a section where the bending moment is 1,667,000 in.-lb. and the shear is 165,000 lb.

Solution.—The method which assumes that one-eighth the area of the web is available as flange area will be used. The formula for stress in the outermost rivet is

$$r = \sqrt{\left(\frac{V}{2n}\right)^2 + \left(\frac{M' \cdot d_n}{2\Sigma d^2}\right)^2} \qquad (14)$$

V = total shear at the section.
M' = moment carried by web.
$2n$ = number of rivets on one side of the splice.
$2\Sigma d^2$ = the sum of the squares of the distances of the rivets, on one side of the splice, from the neutral axis.

The joint must first be designed and then investigated. The number of rivets required is several rivets in excess of the number required to carry the direct shear. The number of ⅞ in. rivets required for shear alone is determined by bearing on the ½ in. web plate, and is

$$2n = \frac{V}{r} = \frac{165{,}000}{10{,}500} = 15.6, \text{ (Table 114).}$$

A joint with 17 rivets spaced as shown in Fig. 16 will be assumed. An odd number of rivets simplifies the calculation.

$$V = 165{,}000 \text{ lb.}$$
$$M' = 1{,}667{,}000 \times 3.00 \div 12.50 = 400{,}000 \text{ in.-lb.}$$
$$2n = 17.$$
$$d_n = 16 \text{ in.}$$
$$2\Sigma d^2 = 2(2^2 + 4^2 + 6^2 + 8^2 + 10^2 + 12^2 + 14^2 + 16^2) = 1632 \text{ in.}^2$$

Then the maximum stress on the outside rivet will be,

$$r = \sqrt{\left(\frac{165\,000}{17}\right)^2 + \left(\frac{400{,}000 \times 16}{1{,}632}\right)^2} = \sqrt{9{,}660^2 + 3{,}920^2} = 10{,}430 \text{ lb.}$$

The allowable value of r for a ⅞ in. rivet is 14,400 lb. in double shear and 10,500 lb. in bearing on ½ in. web plate (Table 114), so the joint is satisfactory.

Net area of flange angles	=	9.50 in.^{2}
One-eighth of area of web plate	=	3.00 "
Total flange area	=	12.50 "

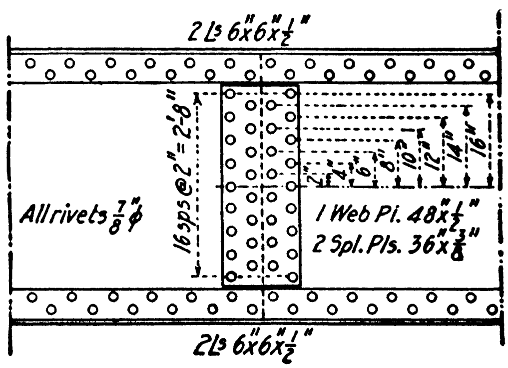


FIG. 16. DETAILS OF A WEB SPLICE.

Riveted Joints in Cylinder, Pipe or Tank.—A cylinder 46 in. in diameter is to be designed to carry an internal pressure of 100 lb. per sq. in. Compute the required thickness of plate and design a longitudinal double riveted lap joint of equal efficiency for all parts. Reduce to commercial dimensions and investigate.

Solution.—The unit stresses allowed by specifications for tanks are f_t = 12,000 lb. per sq. in., f_v = 12,000 lb. per sq. in., f_c = 24,000 lb. per sq. in., for shop joints.

From "Structural Mechanics," Chapter XVI.

$$e = \frac{2f_c}{f_t + 2f_c} = \frac{2 \times 24{,}000}{12{,}000 + 2 \times 24{,}000} = 0.80 \tag{16a}$$

$$t = \frac{w \cdot D}{2f_t \cdot e} = \frac{100 \times 46}{2 \times 12{,}000 \times 0.80} = 0.24 \text{ in.} \tag{16b}$$

$$d = \frac{4f_c}{\pi \cdot fv} \cdot t = \frac{4 \times 24{,}000}{3.1416 \times 12{,}000} \times .24 = 0.61 \text{ in.} \tag{16c}$$

$$p = \left[1 + \frac{2f_c}{f_v}\right] d = \left[1 + \frac{2 \times 24{,}000}{12{,}000}\right] \times 0.61 = 3.05 \text{ in.} \tag{16d}$$

This joint would have the efficiencies for tension, compression and shear all equal, but the sizes could not be obtained from stock so that the joint must be altered to suit commercial sizes.

Make t = ¼ in., d = ⅝ in., p = 3 in., and investigate the joint.

$$P = \frac{w \cdot D \cdot p}{2} = \frac{100 \times 46 \times 3}{2} = 6{,}900 \text{ lb.} \tag{14d}$$

$$f_t = \frac{P}{(p - d)t} = \frac{6{,}900}{2.375 \times 0.25} = 11{,}600 \text{ lb. per sq. in.} \tag{14a}$$

$$f_c = \frac{P}{2t \cdot d} = \frac{6{,}900}{2 \times 0.25 \times 0.625} = 22{,}100 \text{ lb. per sq. in.} \tag{14b}$$

$$f_v = \frac{P}{\frac{1}{4}\pi d^2} = \frac{6{,}900}{0.614} = 11{,}200 \text{ lb. per sq. in.} \tag{14c}$$

Other considerations such as water-tightness enter into the design of joints; see Table 113. Table II*a*, page 452 gives the properties of water tight joints. By efficiency is meant the ratio of the strength of the joint to the strength of a plate of equal thickness. Under effective section of plates in Table II*a*, page 452, is given the thickness of an unriveted plate which would have the same strength as the joint.

The most efficient joint for a given thickness of plate is found as follows: For single riveted lap joint in a ¼ in. plate,

$$d = \frac{4f_c}{\pi \cdot fv} \cdot t = \frac{4 \times 24{,}000}{3.14 \times 12{,}000} \times 0.25 = 0.637 \text{ in.} \tag{15e}$$

$$p = \left[1 + \frac{f_c}{f_t}\right] d = \left[1 + \frac{24{,}000}{12{,}000}\right] d = 3.0d = 1.911 \text{ in.} \tag{15f}$$

$$e = \frac{p - d}{p} = 0.67.$$

Use ⅝ in. rivets with 2 in. pitch.

Formulas for Riveted Joints.—The general formulas for the investigation of lap joints with any number of rows of rivets are (For Nomenclature, see Chapter XVI.),

$$f_t = \frac{P}{(p - d)t}; \quad f_c = \frac{P}{k \cdot t \cdot d}; \quad f_v = \frac{P}{k \cdot \frac{1}{4}\pi \cdot d^2} \tag{28}$$

For design of a joint of maximum efficiency,

$$e = \frac{k \cdot f_c}{f_t + k \cdot fc}; \quad t = \frac{w \cdot D}{2f_t \cdot e}; \quad d = \frac{4f_c}{\pi \cdot fv} \cdot t; \quad p = \left[1 + k\frac{f_c}{f_t}\right] d, \tag{29}$$

where k = number of rows of rivets.

For a butt joint with a single strap plate and a single row of rivets the joint becomes two single riveted lap joints and the formulas for riveted lap joints may be used (Structural Mechanics

13 and 15). For a butt joint with double strap plates and a single row of rivets on each side,

$$f_t = \frac{P}{(p - d)t}; \quad f_c = \frac{P}{t \cdot d}; \quad f_v = \frac{P}{\frac{1}{4}\pi \cdot d^2}. \tag{30}$$

For a butt joint with double strap plates and double riveting on each side,

$$f_t = \frac{P}{(p - d)t}; \quad f_c = \frac{P}{2t \cdot d}; \quad f_v = \frac{P}{\pi \cdot d^2}. \tag{31}$$

When a single strap plate is used it should never be thinner than the main plate, and when double strap plates are used they should never be thinner than ½ the thickness of the main plate.

For data on riveted joints for tanks and stand-pipes, see Table II*a*, page 452.

DESIGN OF LACING BARS FOR COLUMNS.—It is difficult to calculate the bending stresses in a built-up column, and since the shearing stresses depend on the bending stresses the design of lacing bars must be largely a matter of judgment until sufficient tests are made to establish empirical formulas. The following method gives results that agree with tests and with good practice.

For a column with a concentric loading, experiments show that the allowable unit stress may be represented by the straight line formula, $p = 16{,}000 - 70\,l/r$ lb. per sq. in., where p = allowable unit stress in the member; l = length of the member, c. to c. of end connections, and r = radius of gyration of the column, both in inches. Now the allowable unit stress on a short block is 16,000 lb. per sq. in., and the $70\,l/r$ represents the increase in the fiber stress in the column. Now if we assume that this fiber stress is caused by a uniform horizontal load, W, then $\frac{W \cdot l}{8} = \frac{70I \cdot l}{r \cdot c}$, where I = moment of inertia of the cross-section of the column $= A \cdot r^2$, where A = the area of the cross-section of the column, and c = the distance from the neutral axis of column to the extreme fiber in the plane parallel to the plane of the lacing bars. Then $\frac{W \cdot l}{8} = \frac{70A \cdot r^2 \cdot l}{r \cdot c}$, and $W = 560\frac{A \cdot r}{c}$ Now the shear in the column will be $S = W/2$, and the shear is $S = 280\frac{A \cdot r}{c}$, and the stress in a lacing bar will be $= 280\frac{A \cdot r}{c} \times csc\ \theta$, where θ = the angle made by the bar with the axis of the column. In a laced channel column the shearing stress above will be taken by two lacing bars. This shows that the stresses in the lacing bars in the column with a concentric loading depend upon the make-up of the column, and are independent of the length of the column.

Mr. C. C. Schneider by a somewhat different method has deduced the same formula on page 195 of the Report of the Royal Commission on Collapse of Quebec Bridge, 1908.

If the column carries a direct shear in addition to the shear due to the concentric load, or if the column has an eccentric load the additional shearing stresses must be considered in designing the lacing. The total stress in the lacing bar will be the total shear at the section multiplied by the cosec of the angle made by the lacing bar with the axis of the column.

The following formula for stresses in lacing bars is given in the Specifications for Railway Bridge Superstructure, prepared by the Special Committee for Bridge Design and Construction, Am. Soc. C. E., printed in Trans. Am. Soc. C. E., Vol. 86, p. 481.

315.—The latticing of compression members shall be proportioned to resist shearing stress normal to the member not less than that calculated by the formula:

$$R = \frac{P \cdot l}{4{,}000y} \tag{32}$$

in which,

R = normal shearing stress, in pounds;

P = strength of column as a compression member, expressed in pounds;

l = length of column in inches;

y = distance from neutral axis to extreme fiber, in inches.

This formula is based on Rankine's column formula and was deduced by Mr. Otis E. Hovey M. Am. Soc. C. E., Trans. Am. Soc. C. E., Vol. 86, p. 574.

With the same notations for the A.R.E.A., 1910 column formula, $p = 16{,}000 - 70l/r$, deduced as above

$$R = 280A \cdot r/y \tag{33}$$

For the A.R.E.A., 1920 column formula, $p = 15{,}000 - 55l/r$,

$$R = 200A \cdot r/y \tag{34}$$

For all formulas the stress in a lacing bar in a column laced on two sides will be

$$S_1 = \tfrac{1}{2}R \cdot \csc\theta \tag{35}$$

where is the angle between the lacing bar and the axis of the column.

For double lacing on both sides of the stress in a lacing bar will be

$$S_2 = \tfrac{1}{4}R \cdot \csc\theta \tag{36}$$

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References are to Pages in Part I.

STRUCTURAL ENGINEERS' HANDBOOK

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STRUCTURAL TABLES

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STRUCTURAL ENGINEERS' HANDBOOK

PART II.

STRUCTURAL TABLES.

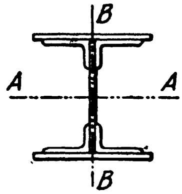
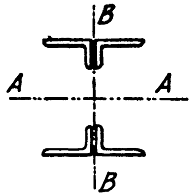
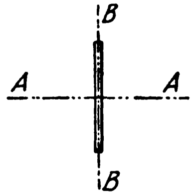
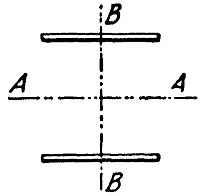
Introduction.—The tables in Part II include the properties of simple rolled sections; the properties of compound sections; the properties of built-up sections for columns, struts and chords; safe loads for angles, beams and channels, and of angle struts; properties of rivets and riveted joints, and miscellaneous data for structural design. It has been the aim to give tables and data that will be of use to the designing engineer and to the student in the designing room rather than to give safe loads, stresses and other predigested data that may be used by the novice. To this end properties of sections are given while safe loads for columns and chords have been omitted. Tables of trigonometric functions and logarithms and other tables that are readily available have not been included. The tables are arranged so that each page is self-contained and self-explanatory. In the tables the properties of rolled sections are grouped together for ease in reference, and are followed by properties of built-up sections. The tables in Part II are numbered in Arabic numerals.

Original Tables.—Tables 3, 4, 5, 13, 19, 20, 21, 22, 32, 33, 34, 35, 36, 37, 38, 39, 40, 56, 57, 58, 59, 60, 61, 62, 63, 64, 65, 66, 67, 68, 69, 70, 71, 72, 73, 74, 78, 79, 80, 81, 82, 83, 84, 85, 86, 87, 134, 135 and 136, covering 136 pages, were calculated especially for this book. The tables have been calculated and checked with great care and are believed to be accurate. These tables are fully protected by copyright and are not to be copied without permission from the author.

The properties of compound sections consisting of two or four angles or of two channels, placed in different relative positions, may be used in designing struts, columns or chords where the sections are held together by means of lacing and tie plates; or the properties of built-up sections may be obtained by combining the moments of inertia of the compound sections and the moments of inertia of one or two plates in the proper relative positions. The built-up sections are all designed to comply with standard specifications and with the standards of the American Bridge Co. for rivet spacing and structural details. To illustrate the use of the tables of compound sections in building up struts, columns and chords, a one page table is given for each built-up section in common use, in which the properties for the usual proportions are given and the methods for calculating additional values by using the key tables of compound sections are given. The method of calculating the properties of built-up sections by using the moments of inertia of compound sections is shown in Table I.

STANDARD TABLES.—The other tables in Part II have been taken from Carnegie Steel Company's "Pocket Companion," Cambria "Steel," American Bridge Company's "Book of Standards," and other sources to which credit has been given. Many of the copied tables have been rearranged and extended. The properties of I-Beams in Table 7, properties of channels in Table 14, and properties of angles in Table 23 and Table 24 were taken from American Bridge Company's "Book of Standards," but have been checked with the recent edition of Carnegie's "Pocket Companion."

TABLE I.

I+II+III	I	II	III
			
Required A Required I_A Required I_B	A of 4Ls Table 33. I_A of 4Ls $= I_X$, Table 33. I_B of 4Ls $= I_Y$, Table 36.	A of Pl. Table 1. I_A of Pl. $= I_1$, Table 3. I_B of Pl. $= I_2$, Table 4.	A of 2Pl. Table 1. I_A of 2Pl. $= I_X$, Table 5. I_B of 2Pl. $= I_1$, Table 3.

I_A = Moment of Inertia, Axis A-A.
I_B = Moment of Inertia, Axis B-B.
r_A = Radius of Gyration, Axis A-A.
r_B = Radius of Gyration, Axis B-B.
I_X = Moment of Inertia, Axis X-X.
I_Y = Moment of Inertia, Axis Y-Y.
I_1 = Moment of Inertia, Axis 1-1.
I_2 = Moment of Inertia, Axis 2-2.
A = Area.
$r_A = \sqrt{\text{Total } I_A \div \text{Total } A}$.
$r_B = \sqrt{\text{Total } I_B \div \text{Total } A}$.

TOP CHORD SECTIONS.—The top chord sections given in Tables 82 to 86 were calculated to comply with the standard specifications which follow, unless otherwise noted in the tables.

Specifications.—All top chord sections shall comply with the following requirements.

Thickness of Metal.—The minimum thickness of metal shall be ¼ in. for highway bridges and ⅜ in. for railway bridges.

Cover Plates.—The cover plate shall have a thickness not less than one-fortieth ($\frac{1}{40}$) the distance between gage lines of rivets in the flange angles on each side of the section. The cover plate shall always have the minimum thickness that will comply with the above requirements.

Web Plates.—The web plates shall have a thickness not less than one-thirtieth ($\frac{1}{30}$) the distance between gage lines of rivets in the flange angles in the line of stress. As much of the metal as practicable shall be concentrated in the web plates and flange angles.

Proportions of Chord Section.—There shall be a top cover plate which shall have a minimum thickness permitted by the specifications. As much of the metal as possible shall be concentrated in the web plates and flange angles. The top and bottom angles shall be so selected as to bring the neutral axis of the section as near the center of the web plates as practicable. The moments of inertia of the section about the two rectangular axes shall be approximately equal.

Note in Third Edition.—The Association of American Steel Manufacturers, in 1923, revised the weights and properties of I-beams and channels as follows: The weights of minimum sections of I-beams and channels were changed to include the weight of the fillets, the areas and other properties remaining unchanged. The weights of other than minimum sections were not changed while the areas were changed to include the fillets, and the other properties were slightly changed.

In adjusting Tables 19 to 22, 57 to 63, and 82 for the changes above, the weights of minimum sections, and the areas of other than minimum sections were changed. All properties for minimum I-beams and channels given in the above tables are, therefore, exact; the properties of other than minimum sections are not exact, but are generally correct within less than two-tenths of one per cent.

STRUCTURAL TABLES.

MISCELLANEOUS SECTIONS.

COLUMNS AND STRUTS.

STRUCTURAL TABLES.

TOP CHORD SECTIONS.

PLATE GIRDERS.

DETAILS FOR BARS.

PINS, BOLTS AND NUTS.

RIVETS AND RIVETING.

STRUCTURAL TABLES.

BEAM AND LATERAL CONNECTIONS.

MISCELLANEOUS.

BETHLEHEM SECTIONS.

STRUCTURAL TABLES.

TABLE 1.

Areas of Bars and Plates.

Square Inches.

Width, Inches.	Thickness, Inches.															
	1/16	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
1/4	.016	.031	.047	.063	.078	.094	.109	.125	.141	.156	.172	.188	.203	.22	.23	.25
1/2	.031	.063	.094	.125	.156	.188	.219	.250	.281	.313	.344	.375	.406	.44	.47	.50
3/4	.047	.094	.141	.188	.234	.281	.328	.375	.422	.469	.516	.563	.609	.66	.70	.75
1	.063	.125	.188	.250	.313	.375	.438	.500	.563	.625	.688	.750	.813	.88	.94	1.00
1 1/4	.078	.156	.234	.313	.391	.469	.547	.625	.703	.781	.859	.938	1.016	1.09	1.17	1.25
1 1/2	.094	.188	.281	.375	.469	.563	.656	.750	.844	.938	1.031	1.125	1.219	1.31	1.41	1.50
1 3/4	.109	.219	.328	.438	.547	.656	.766	.875	.984	1.094	1.203	1.313	1.422	1.53	1.64	1.75
2	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500	1.625	1.75	1.88	2.00
2 1/4	.141	.281	.422	.563	.703	.844	.984	1.125	1.266	1.406	1.547	1.688	1.828	1.97	2.11	2.25
2 1/2	.156	.313	.469	.625	.781	.938	1.094	1.250	1.406	1.563	1.719	1.875	2.031	2.19	2.34	2.50
2 3/4	.172	.344	.516	.688	.859	1.031	1.203	1.375	1.547	1.719	1.891	2.063	2.234	2.41	2.58	2.75
3	.188	.375	.563	.750	.938	1.125	1.313	1.500	1.688	1.875	2.063	2.250	2.438	2.63	2.81	3.00
3 1/4	.203	.406	.609	.813	1.016	1.219	1.422	1.625	1.828	2.031	2.234	2.438	2.641	2.84	3.05	3.25
3 1/2	.219	.438	.656	.875	1.094	1.313	1.531	1.750	1.969	2.188	2.406	2.625	2.844	3.06	3.28	3.50
3 3/4	.234	.469	.703	.938	1.172	1.406	1.641	1.875	2.109	2.344	2.578	2.813	3.047	3.28	3.52	3.75
4	.250	.500	.750	1.000	1.250	1.500	1.750	2.000	2.250	2.500	2.750	3.000	3.250	3.50	3.75	4.00
4 1/4	.266	.531	.797	1.063	1.328	1.594	1.859	2.125	2.391	2.656	2.922	3.188	3.453	3.72	3.98	4.25
4 1/2	.281	.563	.844	1.125	1.406	1.688	1.969	2.250	2.531	2.813	3.094	3.375	3.656	3.94	4.22	4.50
4 3/4	.297	.594	.891	1.188	1.484	1.781	2.078	2.375	2.672	2.969	3.266	3.563	3.859	4.16	4.45	4.75
5	.313	.625	.938	1.250	1.563	1.875	2.188	2.500	2.813	3.125	3.438	3.750	4.063	4.38	4.69	5.00
5 1/4	.328	.656	.984	1.313	1.641	1.969	2.297	2.625	2.953	3.281	3.609	3.938	4.266	4.59	4.92	5.25
5 1/2	.344	.688	1.031	1.375	1.719	2.063	2.406	2.750	3.094	3.438	3.781	4.125	4.469	4.81	5.16	5.50
5 3/4	.359	.719	1.078	1.438	1.797	2.156	2.516	2.875	3.234	3.594	3.953	4.313	4.672	5.03	5.39	5.75
6	.375	.750	1.125	1.500	1.875	2.250	2.625	3.000	3.375	3.750	4.125	4.500	4.875	5.25	5.63	6.00
6 1/4	391	.781	1.172	1.563	1.953	2.344	2.734	3.125	3.516	3.906	4.297	4.688	5.078	5.47	5.86	6.25
6 1/2	.406	.813	1.219	1.625	2.031	2.438	2.844	3.250	3.656	4.063	4.469	4.875	5.281	5.69	6.09	6.50
6 3/4	.422	.844	1.266	1.688	2.109	2.531	2.953	3.375	3.797	4.219	4.641	5.063	5.484	5.91	6.33	6.75
7	.438	.875	1.313	1.750	2.188	2.625	3.063	3.500	3.938	4.375	4.813	5.250	5.688	6.13	6.56	7.00
7 1/4	.453	.906	1.359	1.813	2.266	2.719	3.172	3.625	4.078	4.531	4.984	5.438	5.891	6.34	6.80	7.25
7 1/2	.469	.938	1.406	1.875	2.344	2.813	3.281	3.750	4.219	4.688	5.156	5.625	6.094	6.56	7.03	7.50
7 3/4	.484	.969	1.453	1.938	2.422	2.906	3.391	3.875	4.359	4.844	5.328	5.813	6.297	6.78	7.27	7.75
8	.500	1.000	1.500	2.000	2.500	3.000	3.500	4.000	4.500	5.000	5.500	6.000	6.500	7.00	7.50	8.00
8 1/4	.516	1.031	1.547	2.063	2.578	3.094	3.609	4.125	4.641	5.156	5.672	6.188	6.703	7.22	7.73	8.25
8 1/2	.531	1.063	1.594	2.125	2.656	3.188	3.719	4.250	4.781	5.313	5.844	6.375	6.906	7.44	7.97	8.50
8 3/4	.547	1.094	1.641	2.188	2.734	3.281	3.828	4.375	4.922	5.469	6.016	6.563	7.109	7.66	8.20	8.75
9	.563	1.125	1.688	2.250	2.813	3.375	3.938	4.500	5.063	5.625	6.188	6.750	7.313	7.88	8.44	9.00
9 1/4	.578	1.156	1.734	2.313	2.891	3.469	4.047	4.625	5.203	5.781	6.359	6.938	7.516	8.09	8.67	9.25
9 1/2	.594	1.188	1.781	2.375	2.969	3.563	4.156	4.750	5.344	5.938	6.531	7.125	7.719	8.31	8.91	9.50
9 3/4	.609	1.219	1.828	2.438	3.047	3.656	4.266	4.875	5.484	6.094	6.703	7.313	7.922	8.53	9.14	9.75
10	.625	1.250	1.875	2.500	3.125	3.750	4.375	5.000	5.625	6.250	6.875	7.500	8.125	8.75	9.38	10.00
10 1/4	.641	1.281	1.922	2.563	3.203	3.844	4.484	5.125	5.766	6.406	7.047	7.688	8.328	8.97	9.61	10.25
10 1/2	.656	1.313	1.969	2.625	3.281	3.938	4.594	5.250	5.906	6.563	7.219	7.875	8.531	9.19	9.84	10.50
10 3/4	.672	1.344	2.016	2.688	3.359	4.031	4.703	5.375	6.047	6.719	7.391	8.063	8.734	9.41	10.08	10.75
11	.688	1.375	2.063	2.750	3.438	4.125	4.813	5.500	6.188	6.875	7.563	8.250	8.938	9.63	10.31	11.00
11 1/4	.703	1.406	2.109	2.813	3.516	4.219	4.922	5.625	6.328	7.031	7.734	8.438	9.141	9.84	10.55	11.25
11 1/2	.719	1.438	2.156	2.875	3.594	4.313	5.031	5.750	6.469	7.188	7.906	8.625	9.344	10.06	10.78	11.50
11 3/4	.734	1.469	2.203	2.938	3.672	4.406	5.141	5.875	6.609	7.344	8.078	8.813	9.547	10.28	11.02	11.75
12	.750	1.500	2.250	3.000	3.750	4.500	5.250	6.000	6.750	7.500	8.250	9.000	9.750	10.50	11.25	12.00

TABLE 1.—*Continued.*

Areas of Bars and Plates.

Square Inches.

Width, Inches.	Thickness, Inches. 1/16	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
12½	.781	1.563	2.344	3.13	3.91	4.69	5.47	6.25	7.03	7.81	8.59	9.38	10.16	10.94	11.72	12.50
13	.813	1.625	2.438	3.25	4.06	4.88	5.69	6.50	7.31	8.13	8.94	9.75	10.56	11.38	12.19	13.00
13½	.844	1.688	2.531	3.38	4.22	5.06	5.91	6.75	7.59	8.44	9.28	10.13	10.97	11.81	12.66	13.50
14	.875	1.750	2.625	3.50	4.38	5.25	6.13	7.00	7.88	8.75	9.63	10.50	11.38	12.25	13.13	14.00
14½	.906	1.813	2.719	3.63	4.53	5.44	6.34	7.25	8.16	9.06	9.97	10.88	11.78	12.69	13.59	14.50
15	.938	1.875	2.813	3.75	4.69	5.63	6.56	7.50	8.44	9.38	10.31	11.25	12.19	13.13	14.06	15.00
15½	.969	1.938	2.906	3.88	4.84	5.81	6.78	7.75	8.72	9.69	10.66	11.63	12.59	13.56	14.53	15.50
16	1.000	2.000	3.000	4.00	5.00	6.00	7.00	8.00	9.00	10.00	11.00	12.00	13.00	14.00	15.00	16.00
16½	1.031	2.063	3.094	4.13	5.16	6.19	7.22	8.25	9.28	10.31	11.34	12.38	13.41	14.44	15.47	16.50
17	1.063	2.125	3.188	4.25	5.31	6.38	7.44	8.50	9.56	10.63	11.69	12.75	13.81	14.88	15.94	17.00
17½	1.094	2.188	3.281	4.38	5.47	6.56	7.66	8.75	9.84	10.94	12.03	13.13	14.22	15.31	16.41	17.50
18	1.125	2.250	3.375	4.50	5.63	6.75	7.88	9.00	10.13	11.25	12.38	13.50	14.63	15.75	16.88	18.00
18½	1.156	2.313	3.469	4.63	5.78	6.94	8.09	9.25	10.41	11.56	12.72	13.88	15.03	16.19	17.34	18.50
19	1.188	2.375	3.563	4.75	5.94	7.13	8.31	9.50	10.69	11.88	13.06	14.25	15.44	16.63	17.81	19.00
19½	1.219	2.438	3.656	4.88	6.09	7.31	8.53	9.75	10.97	12.19	13.41	14.63	15.84	17.06	18.28	19.50
20	1.250	2.500	3.750	5.00	6.25	7.50	8.75	10.00	11.25	12.50	13.75	15.00	16.25	17.50	18.75	20.00
20½	1.281	2.563	3.844	5.13	6.41	7.69	8.97	10.25	11.53	12.81	14.09	15.38	16.66	17.94	19.22	20.50
21	1.313	2.625	3.938	5.25	6.56	7.88	9.19	10.50	11.81	13.13	14.44	15.75	17.06	18.38	19.69	21.00
21½	1.344	2.688	4.031	5.38	6.72	8.06	9.41	10.75	12.09	13.44	14.78	16.13	17.47	18.81	20.16	21.50
22	1.375	2.750	4.125	5.50	6.88	8.25	9.63	11.00	12.38	13.75	15.13	16.50	17.88	19.25	20.63	22.00
22½	1.406	2.813	4.219	5.63	7.03	8.44	9.84	11.25	12.66	14.06	15.47	16.88	18.28	19.69	21.09	22.50
23	1.438	2.875	4.313	5.75	7.19	8.63	10.06	11.50	12.94	14.38	15.81	17.25	18.69	20.13	21.56	23.00
23½	1.469	2.938	4.406	5.88	7.34	8.81	10.28	11.75	13.22	14.69	16.16	17.63	19.09	20.56	22.03	23.50
24	1.500	3.000	4.500	6.00	7.50	9.00	10.50	12.00	13.50	15.00	16.50	18.00	19.50	21.00	22.50	24.00
25	1.563	3.125	4.688	6.25	7.81	9.38	10.94	12.50	14.06	15.63	17.19	18.75	20.31	21.88	23.44	25.00
26	1.625	3.250	4.875	6.50	8.13	9.75	11.38	13.00	14.63	16.25	17.88	19.50	21.13	22.75	24.38	26.00
27	1.688	3.375	5.063	6.75	8.44	10.13	11.81	13.50	15.19	16.88	18.56	20.25	21.94	23.63	25.31	27.00
28	1.750	3.500	5.250	7.00	8.75	10.50	12.25	14.00	15.75	17.50	19.25	21.00	22.75	24.50	26.25	28.00
29	1.813	3.625	5.438	7.25	9.06	10.88	12.69	14.50	16.31	18.13	19.94	21.75	23.56	25.38	27.19	29.00
30	1.875	3.750	5.625	7.50	9.38	11.25	13.13	15.00	16.88	18.75	20.63	22.50	24.38	26.25	28.13	30.00
31	1.938	3.875	5.813	7.75	9.69	11.63	13.56	15.50	17.44	19.38	21.31	23.25	25.19	27.13	29.06	31.00
32	2.000	4.000	6.000	8.00	10.00	12.00	14.00	16.00	18.00	20.00	22.00	24.00	26.00	28.00	30.00	32.00
33	2.063	4.125	6.188	8.25	10.31	12.38	14.44	16.50	18.56	20.63	22.69	24.75	26.81	28.88	30.94	33.00
34	2.125	4.250	6.375	8.50	10.63	12.75	14.88	17.00	19.13	21.25	23.38	25.50	27.63	29.75	31.88	34.00
35	2.188	4.375	6.563	8.75	10.94	13.13	15.31	17.50	19.69	21.88	24.06	26.25	28.44	30.63	32.81	35.00
36	2.250	4.500	6.750	9.00	11.25	13.50	15.75	18.00	20.25	22.50	24.75	27.00	29.25	31.50	33.75	36.00
37	2.313	4.625	6.938	9.25	11.56	13.88	16.19	18.50	20.81	23.13	25.44	27.75	30.06	32.38	34.69	37.00
38	2.375	4.750	7.125	9.50	11.88	14.25	16.63	19.00	21.38	23.75	26.13	28.50	30.88	33.25	35.63	38.00
39	2.438	4.875	7.313	9.75	12.19	14.63	17.06	19.50	21.94	24.38	26.81	29.25	31.69	34.13	36.56	39.00
40	2.500	5.000	7.500	10.00	12.50	15.00	17.50	20.00	22.50	25.00	27.50	30.00	32.50	35.00	37.50	40.00
41	2.563	5.125	7.688	10.25	12.81	15.38	17.94	20.50	23.06	25.63	28.19	30.75	33.31	35.88	38.44	41.00
42	2.625	5.250	7.875	10.50	13.13	15.75	18.38	21.00	23.63	26.25	28.88	31.50	34.13	36.75	39.38	42.00
43	2.688	5.375	8.063	10.75	13.44	16.13	18.81	21.50	24.19	26.88	29.56	32.25	34.94	37.63	40.31	43.00
44	2.750	5.500	8.250	11.00	13.75	16.50	19.25	22.00	24.75	27.50	30.25	33.00	35.75	38.50	41.25	44.00
45	2.813	5.625	8.438	11.25	14.06	16.88	19.69	22.50	25.31	28.13	30.94	33.75	36.56	39.38	42.19	45.00
46	2.875	5.750	8.625	11.50	14.38	17.25	20.13	23.00	25.88	28.75	31.63	34.50	37.38	40.25	43.13	46.00
47	2.938	5.875	8.813	11.75	14.69	17.63	20.56	23.50	26.44	29.38	32.31	35.25	38.19	41.13	44.06	47.00
48	3.000	6.000	9.000	12.00	15.00	18.00	21.00	24.00	27.00	30.00	33.00	36.00	39.00	42.00	45.00	48.00

TABLE 1.—*Continued.*

Areas of Bars and Plates.

Square Inches.

Width, Inches.	Thickness, Inches.															
	1/16	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
49	3.06	6.13	9.19	12.25	15.31	18.38	21.44	24.50	27.56	30.63	33.69	36.75	39.81	42.88	45.94	49.00
50	3.13	6.25	9.38	12.50	15.63	18.75	21.88	25.00	28.13	31.25	34.38	37.50	40.63	43.75	46.88	50.00
51	3.19	6.38	9.56	12.75	15.94	19.13	22.31	25.50	28.69	31.88	35.06	38.25	41.44	44.63	47.81	51.00
52	3.25	6.50	9.75	13.00	16.25	19.50	22.75	26.00	29.25	32.50	35.75	39.00	42.25	45.50	48.75	52.00
53	3.31	6.63	9.94	13.25	16.56	19.88	23.19	26.50	29.81	33.13	36.44	39.75	43.06	46.38	49.69	53.00
54	3.38	6.75	10.13	13.50	16.88	20.25	23.63	27.00	30.38	33.75	37.13	40.50	43.88	47.25	50.63	54.00
55	3.44	6.88	10.31	13.75	17.19	20.63	24.06	27.50	30.94	34.38	37.81	41.25	44.69	48.13	51.56	55.00
56	3.50	7.00	10.50	14.00	17.50	21.00	24.50	28.00	31.50	35.00	38.50	42.00	45.50	49.00	52.50	56.00
57	3.56	7.13	10.69	14.25	17.81	21.38	24.94	28.50	32.06	35.63	39.19	42.75	46.31	49.88	53.44	57.00
58	3.63	7.25	10.88	14.50	18.13	21.75	25.38	29.00	32.63	36.25	39.88	43.50	47.13	50.75	54.38	58.00
59	3.69	7.38	11.06	14.75	18.44	22.13	25.81	29.50	33.19	36.88	40.56	44.25	47.94	51.63	55.31	59.00
60	3.75	7.50	11.25	15.00	18.75	22.50	26.25	30.00	33.75	37.50	41.25	45.00	48.75	52.50	56.25	60.00
61	3.81	7.63	11.44	15.25	19.06	22.88	26.69	30.50	34.31	38.13	41.94	45.75	49.56	53.38	57.19	61.00
62	3.88	7.75	11.63	15.50	19.38	23.25	27.13	31.00	34.88	38.75	42.63	46.50	50.38	54.25	58.13	62.00
63	3.94	7.88	11.81	15.75	19.69	23.63	27.56	31.50	35.44	39.38	43.31	47.25	51.19	55.13	59.06	63.00
64	4.00	8.00	12.00	16.00	20.00	24.00	28.00	32.00	36.00	40.00	44.00	48.00	52.00	56.00	60.00	64.00
65	4.06	8.13	12.19	16.25	20.31	24.38	28.44	32.50	36.56	40.63	44.69	48.75	52.81	56.88	60.94	65.00
66	4.13	8.25	12.38	16.50	20.63	24.75	28.88	33.00	37.13	41.25	45.38	49.50	53.63	57.75	61.88	66.00
67	4.19	8.38	12.56	16.75	20.94	25.13	29.31	33.50	37.69	41.88	46.06	50.25	54.44	58.63	62.81	67.00
68	4.25	8.50	12.75	17.00	21.25	25.50	29.75	34.00	38.25	42.50	46.75	51.00	55.25	59.50	63.75	68.00
69	4.31	8.63	12.94	17.25	21.56	25.88	30.19	34.50	38.81	43.13	47.44	51.75	56.06	60.38	64.69	69.00
70	4.38	8.75	13.13	17.50	21.88	26.25	30.63	35.00	39.38	43.75	48.13	52.50	56.88	61.25	65.63	70.00
71	4.44	8.88	13.31	17.75	22.19	26.63	31.06	35.50	39.94	44.38	48.81	53.25	57.69	62.13	66.56	71.00
72	4.50	9.00	13.50	18.00	22.50	27.00	31.50	36.00	40.50	45.00	49.50	54.00	58.50	63.00	67.50	72.00
73	4.56	9.13	13.69	18.25	22.81	27.38	31.94	36.50	41.06	45.63	50.19	54.75	59.31	63.88	68.44	73.00
74	4.63	9.25	13.88	18.50	23.13	27.75	32.38	37.00	41.63	46.25	50.88	55.50	60.13	64.75	69.38	74.00
75	4.69	9.38	14.06	18.75	23.44	28.13	32.81	37.50	42.19	46.88	51.56	56.25	60.94	65.63	70.31	75.00
76	4.75	9.50	14.25	19.00	23.75	28.50	33.25	38.00	42.75	47.50	52.25	57.00	61.75	66.50	71.25	76.00
77	4.81	9.63	14.44	19.25	24.06	28.88	33.69	38.50	43.31	48.13	52.94	57.75	62.56	67.38	72.19	77.00
78	4.88	9.75	14.63	19.50	24.38	29.25	34.13	39.00	43.88	48.75	53.63	58.50	63.38	68.25	73.13	78.00
79	4.94	9.88	14.81	19.75	24.69	29.63	34.56	39.50	44.44	49.38	54.31	59.25	64.19	69.13	74.06	79.00
80	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00	45.00	50.00	55.00	60.00	65.00	70.00	75.00	80.00
81	5.06	10.13	15.19	20.25	25.31	30.38	35.44	40.50	45.56	50.63	55.69	60.75	65.81	70.88	75.94	81.00
82	5.13	10.25	15.38	20.50	25.63	30.75	35.88	41.00	46.13	51.25	56.38	61.50	66.63	71.75	76.88	82.00
83	5.19	10.38	15.56	20.75	25.94	31.13	36.31	41.50	46.69	51.88	57.06	62.25	67.44	72.63	77.81	83.00
84	5.25	10.50	15.75	21.00	26.25	31.50	36.75	42.00	47.25	52.50	57.75	63.00	68.25	73.50	78.75	84.00
85	5.31	10.63	15.94	21.25	26.56	31.88	37.19	42.50	47.81	53.13	58.44	63.75	69.06	74.38	79.69	85.00
86	5.38	10.75	16.13	21.50	26.88	32.25	37.63	43.00	48.38	53.75	59.13	64.50	69.88	75.25	80.63	86.00
87	5.44	10.88	16.31	21.75	27.19	32.63	38.06	43.50	48.94	54.38	59.81	65.25	70.69	76.13	81.56	87.00
88	5.50	11.00	16.50	22.00	27.50	33.00	38.50	44.00	49.50	55.00	60.50	66.00	71.50	77.00	82.50	88.00
89	5.56	11.13	16.69	22.25	27.81	33.38	38.94	44.50	50.06	55.63	61.19	66.75	72.31	77.88	83.44	89.00
90	5.63	11.25	16.88	22.50	28.13	33.75	39.38	45.00	50.63	56.25	61.88	67.50	73.13	78.75	84.38	90.00
91	5.69	11.38	17.06	22.75	28.44	34.13	39.81	45.50	51.19	56.88	62.56	68.25	73.94	79.63	85.31	91.00
92	5.75	11.50	17.25	23.00	28.75	34.50	40.25	46.00	51.75	57.50	63.25	69.00	74.75	80.50	86.25	92.00
93	5.81	11.63	17.44	23.25	29.06	34.88	40.69	46.50	52.31	58.13	63.94	69.75	75.56	81.38	87.19	93.00
94	5.88	11.75	17.63	23.50	29.38	35.25	41.13	47.00	52.88	58.75	64.63	70.50	76.38	82.25	88.13	94.00
95	5.94	11.88	17.81	23.75	29.69	35.63	41.56	47.50	53.44	59.38	65.31	71.25	77.19	83.13	89.06	95.00
96	6.00	12.00	18.00	24.00	30.00	36.00	42.00	48.00	54.00	60.00	66.00	72.00	78.00	84.00	90.00	96.00
97	6.06	12.13	18.19	24.25	30.31	36.38	42.44	48.50	54.56	60.63	66.69	72.75	78.81	84.88	90.94	97.00
98	6.13	12.25	18.38	24.50	30.63	36.75	42.88	49.00	55.13	61.25	67.38	73.50	79.63	85.75	91.88	98.00
99	6.19	12.38	18.56	24.75	30.94	37.13	43.31	49.50	55.69	61.88	68.06	74.25	80.44	86.63	92.81	99.00
100	6.25	12.50	18.75	25.00	31.25	37.50	43.75	50.00	56.25	62.50	68.75	75.00	81.25	87.50	93.75	100.0

TABLE 2.

Weights of Steel Bars and Plates.

[redacted]

Pounds per Lineal Foot.

Width, Inches.	Thickness, Inches.															
	1/16	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
1/4	.053	.106	.159	.213	.27	.32	.37	.43	.48	.53	.58	.64	.69	.74	.80	.85
1/2	.106	.213	.319	.425	.53	.64	.74	.85	.96	1.06	1.17	1.28	1.38	1.49	1.59	1.70
3/4	.159	.319	.478	.638	.80	.96	1.12	1.28	1.43	1.59	1.75	1.91	2.07	2.23	2.39	2.55
1	.213	.425	.638	.850	1.06	1.28	1.49	1.70	1.91	2.13	2.34	2.55	2.76	2.98	3.19	3.40
1 1/4	.266	.531	.797	1.063	1.33	1.59	1.86	2.13	2.39	2.66	2.92	3.19	3.45	3.72	3.98	4.25
1 1/2	.319	.638	.956	1.275	1.59	1.91	2.23	2.55	2.87	3.19	3.51	3.83	4.14	4.46	4.78	5.10
1 3/4	.372	.744	1.116	1.488	1.86	2.23	2.60	2.98	3.35	3.72	4.09	4.46	4.83	5.21	5.58	5.95
2	.425	.850	1.275	1.700	2.13	2.55	2.98	3.40	3.83	4.25	4.68	5.10	5.53	5.95	6.38	6.80
2 1/4	.478	.956	1.434	1.913	2.39	2.87	3.35	3.83	4.30	4.78	5.26	5.74	6.22	6.69	7.17	7.65
2 1/2	.531	1.063	1.594	2.125	2.66	3.19	3.72	4.25	4.78	5.31	5.84	6.38	6.91	7.44	7.97	8.50
2 3/4	.584	1.169	1.753	2.338	2.92	3.51	4.09	4.68	5.26	5.84	6.43	7.01	7.60	8.18	8.77	9.35
3	.638	1.275	1.913	2.550	3.19	3.83	4.46	5.10	5.74	6.38	7.01	7.65	8.29	8.93	9.56	10.20
3 1/4	.691	1.381	2.072	2.763	3.45	4.14	4.83	5.53	6.22	6.91	7.60	8.29	8.98	9.67	10.36	11.05
3 1/2	.744	1.488	2.231	2.975	3.72	4.46	5.21	5.95	6.69	7.44	8.18	8.93	9.67	10.41	11.16	11.90
3 3/4	.797	1.594	2.391	3.188	3.98	4.78	5.58	6.38	7.17	7.97	8.77	9.56	10.36	11.16	11.95	12.75
4	.850	1.700	2.550	3.400	4.25	5.10	5.95	6.80	7.65	8.50	9.35	10.20	11.05	11.90	12.75	13.60
4 1/4	.903	1.806	2.709	3.613	4.52	5.42	6.32	7.23	8.13	9.03	9.93	10.84	11.74	12.64	13.55	14.45
4 1/2	.956	1.913	2.869	3.825	4.78	5.74	6.69	7.65	8.61	9.56	10.52	11.48	12.43	13.39	14.34	15.30
4 3/4	1.009	2.019	3.028	4.038	5.05	6.06	7.07	8.08	9.08	10.09	11.10	12.11	13.12	14.13	15.14	16.15
5	1.063	2.125	3.188	4.250	5.31	6.38	7.44	8.50	9.56	10.63	11.69	12.75	13.81	14.88	15.94	17.00
5 1/4	1.116	2.231	3.347	4.463	5.58	6.69	7.81	8.93	10.04	11.16	12.27	13.39	14.50	15.62	16.73	17.85
5 1/2	1.169	2.338	3.506	4.675	5.84	7.01	8.18	9.35	10.52	11.69	12.86	14.03	15.19	16.36	17.53	18.70
5 3/4	1.222	2.444	3.666	4.888	6.11	7.33	8.55	9.78	11.00	12.22	13.44	14.66	15.88	17.11	18.33	19.55
6	1.275	2.550	.825	5.100	6.38	7.65	8.93	10.20	11.48	12.75	14.03	15.30	16.58	17.85	19.13	20.40
6 1/4	1.328	2.656	3.984	5.313	6.64	7.97	9.30	10.63	11.95	13.28	14.61	15.94	17.27	18.59	19.92	21.25
6 1/2	1.381	2.763	4.144	5.525	6.91	8.29	9.67	11.05	12.43	13.81	15.19	16.58	17.96	19.34	20.72	22.10
6 3/4	1.434	2.869	4.303	5.738	7.17	8.61	10.04	11.48	12.91	14.34	15.78	17.21	18.65	20.08	21.52	22.95
7	1.488	2.975	4.463	5.950	7.44	8.93	10.41	11.90	13.39	14.88	16.36	17.85	19.34	20.83	22.31	23.80
7 1/4	1.541	3.081	4.622	6.163	7.70	9.24	10.78	12.33	13.87	15.41	16.95	18.49	20.03	21.57	23.11	24.65
7 1/2	1.594	3.188	4.781	6.375	7.97	9.56	11.16	12.75	14.34	15.94	17.53	19.13	20.72	22.31	23.91	25.50
7 3/4	1.647	3.294	4.941	6.588	8.23	9.88	11.53	13.18	14.82	16.47	18.12	19.76	21.41	23.06	24.70	26.35
8	1.700	3.400	5.100	6.800	8.50	10.20	11.90	13.60	15.30	17.00	18.70	20.40	22.10	23.80	25.50	27.20
8 1/4	1.753	3.506	5.259	7.013	8.77	10.52	12.27	14.03	15.78	17.53	19.28	21.04	22.79	24.54	26.30	28.05
8 1/2	1.806	3.613	5.419	7.225	9.03	10.84	12.64	14.45	16.26	18.06	19.87	21.68	23.48	25.29	27.09	28.90
8 3/4	1.859	3.719	5.578	7.438	9.30	11.16	13.02	14.88	16.73	18.59	20.45	22.31	24.17	26.03	27.89	29.75
9	1.913	3.825	5.738	7.650	9.56	11.48	13.39	15.30	17.21	19.13	21.04	22.95	24.86	26.78	28.69	30.60
9 1/4	1.966	3.931	5.897	7.863	9.83	11.79	13.76	15.73	17.69	19.66	21.62	23.59	25.55	27.52	29.48	31.45
9 1/2	2.019	4.038	6.056	8.075	10.09	12.11	14.13	16.15	18.17	20.19	22.21	24.23	26.24	28.26	30.28	32.30
9 3/4	2.072	4.144	6.216	8.288	10.36	12.43	14.50	16.58	18.65	20.72	22.79	24.86	26.93	29.01	31.08	33.15
10	2.125	4.250	6.375	8.500	10.63	12.75	14.88	17.00	19.13	21.25	23.38	25.50	27.63	29.75	31.88	34.00
10 1/4	2.178	4.356	6.534	8.713	10.89	13.07	15.25	17.43	19.60	21.78	23.96	26.14	28.32	30.49	32.67	34.85
10 1/2	2.231	4.463	6.694	8.925	11.16	13.39	15.62	17.85	20.08	22.31	24.54	26.78	29.01	31.24	33.47	35.70
10 3/4	2.284	4.569	6.853	9.138	11.42	13.71	15.99	18.28	20.56	22.84	25.13	27.41	29.70	31.98	34.27	36.55
11	2.338	4.675	7.013	9.350	11.69	14.03	16.36	18.70	21.04	23.38	25.71	28.05	30.39	32.73	35.06	37.40
11 1/4	2.391	4.781	7.172	9.563	11.95	14.34	16.73	19.13	21.52	23.91	26.30	28.69	31.08	33.47	35.86	38.25
11 1/2	2.444	4.888	7.331	9.775	12.22	14.66	17.11	19.55	21.99	24.44	26.88	29.33	31.77	34.21	36.66	39.10
11 3/4	2.497	4.994	7.491	9.988	12.48	14.98	17.48	19.98	22.47	24.97	27.47	29.96	32.46	34.96	37.45	39.95
12	2.550	5.100	7.650	10.20	12.75	15.30	17.85	20.40	22.95	25.50	28.05	30.60	33.15	35.70	38.25	40.80

TABLE 2.—*Continued.*

WEIGHTS OF STEEL BARS AND PLATES.

[redacted]

POUNDS PER LINEAL FOOT.

Width, Inches.	Thickness, Inches.															
	1/16	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
12½	2.66	5.31	7.97	10.63	13.28	15.94	18.59	21.25	23.91	26.56	29.2	31 9	34.5	37.2	39.8	42.5
13	2.76	5.53	8.29	11.05	13.81	16.58	19.34	22.10	24.86	27.63	30.4	33.2	35.9	38.7	41.4	44.2
13½	2.87	5.74	8.61	11.48	14.34	17.21	20.08	22.95	25.82	28.69	31.6	34.4	37.3	40.2	43.0	45 9
14	2.98	5.95	8.93	11.90	14.88	17.85	20.83	23.80	26.78	29.75	32.7	35.7	38.7	41.7	44.6	47.6
14½	3.08	6.16	9.24	12.33	15.41	18.49	21.57	24.65	27.73	30.81	33.9	37.0	40.1	43.1	46.2	49.3
15	3.19	6.38	9.56	12.75	15.94	19.13	22.31	25.50	28.69	31.88	35.1	38.3	41.4	44.6	47.8	51.0
15½	3.29	6.59	9.88	13.18	16.47	19.76	23.06	26.35	29.64	32.94	36.2	39.5	42.8	46.1	49.4	52.7
16	3.40	6.80	10.20	13.60	17.00	20.40	23.80	27.20	30.60	34.00	37.4	40.8	44.2	47.6	51 0	54.4
16½	3 51	7.01	10.52	14.03	17.53	21.04	24.54	28.05	31.56	35.06	38.6	42.1	45.6	49.1	52.6	56.1
17	3.61	7.23	10.84	14.45	18.06	21.68	25.29	28.90	32.51	36.13	39.7	43.4	47.0	50.6	54.2	57.8
17½	3.72	7.44	11.16	14.88	18.59	22.31	26.03	29.75	33.47	37 19	40.9	44.6	48.3	52.1	55.8	59.5
18	3.83	7.65	11.48	15.30	19.13	22.95	26.78	30.60	34.43	38.25	42.1	45.9	49.7	53.6	57.4	61.2
18½	3.93	7.86	11.79	15.73	19 66	23.59	27.52	31.45	35.38	39.31	43.2	47.2	51.1	55.0	59.0	62.9
19	4.04	8.08	12.11	16.15	20.19	24.23	28.26	32.30	36.34	40.38	44.4	48.5	52.5	56.5	60.6	64.6
19½	4.14	8.29	12.43	16.58	20.72	24.86	29.01	33.15	37.29	41.44	45.6	49.7	53.9	58.0	62.2	66.3
20	4.25	8.50	12.75	17.00	21.25	25.50	29.75	34.00	38.25	42.50	46.8	51.0	55.3	59.5	63.8	68.0
20½	4.36	8.71	13.07	17.43	21.78	26.14	30.49	34.85	39.21	43.56	47.9	52.3	56.6	61.0	65.3	69.7
21	4.46	8.93	13.39	17.85	22.31	26.78	31.24	35.70	40.16	44.63	49.1	53.6	58.0	62.5	66 9	71.4
21½	4.57	9.14	13.71	18.28	22.84	27.41	31.98	36.55	41.12	45.69	50.3	54.8	59.4	64.0	68.5	73.1
22	4.68	9.35	14.03	18.70	23.38	28.05	32.73	37.40	42.08	46.75	51.4	56.1	60.8	65.5	70.1	74.8
22½	4.78	9.56	14.34	19.13	23.91	28.69	33.47	38.25	43.03	47.81	52.6	57.4	62.2	66.9	71.7	76.5
23	4.89	9.78	14.66	19 55	24.44	29.33	34.21	39.10	43.99	48.88	53.8	58.7	63.5	68.4	73.3	78.2
23½	4.99	9.99	14.98	19.98	24.97	29.96	34.96	39.95	44.94	49.94	54.9	59.9	64.9	69.9	74.9	79.9
24	5.10	10.20	15.30	20.40	25.50	30.60	35.70	40.80	45.90	51.00	56.1	61.2	66.3	71.4	76.5	81.6
25	5.31	10.63	15.94	21.25	26 56	31.88	37.19	42.50	47.81	53.13	58.4	63.8	69.1	74.4	79.7	85.0
26	5.53	11.05	16.58	22.10	27.63	33.15	38.68	44.20	49.73	55.25	60.8	66.3	71.8	77.4	82.9	88.4
27	5.74	11.48	17.21	22.95	28.69	34.43	40.16	45.90	51.64	57.38	63.1	68.9	74.6	80.3	86.1	91.8
28	5.95	11.90	17.85	23.80	29.75	35.70	41.65	47.60	53.55	59.50	65.5	71.4	77.4	83.3	89.3	95.2
29	6.16	12.33	18.49	24.65	30.81	36.98	43.14	49.30	55.46	61.63	67.8	74.0	80.1	86.3	92.4	98.6
30	6.38	12.75	19.13	25.50	31.88	38.25	44.63	51.00	57.38	63.75	70.1	76.5	82.9	89.3	95.6	102.0
31	6.59	13.18	19.76	26.35	32 94	39.53	46.11	52.70	59.29	65.88	72.5	79.1	85.6	92.2	98.8	105.4
32	6.80	13.60	20.40	27.20	34.00	40.80	47.60	54.40	61.20	68.00	74.8	81.6	88.4	95.2	102.0	108.8
33	7.01	14.03	21.04	28.05	35.06	42.08	49.09	56.10	63.11	70.13	77.1	84.2	91.2	98.2	105.2	112.2
34	7.23	14.45	21.68	28 90	36.13	43.35	50.58	57.80	65.03	72.25	79.5	86.7	93.9	101.2	108.4	115.6
35	7.44	14.88	22.31	29.75	37.19	44.63	52.06	59.50	66.94	74.38	81.8	89.3	96.7	104.1	111.6	119.0
36	7.65	15.30	22.95	30.60	38.25	45.90	53.55	61.20	68.85	76.50	84.2	91.8	99.5	107.1	114.8	122.4
37	7.86	15.73	23.59	31.45	39.31	47.18	55.04	62.90	70.76	78.63	86.5	94.4	102.2	110.1	117.9	125.8
38	8.08	16.15	24.23	32.30	40.38	48.45	56.53	64.60	72.68	80.75	88.8	96.9	105.0	113.1	121.1	129.2
39	8.29	16.58	24.86	33.15	41.44	49.73	58.01	66.30	74.59	82.88	91.2	99.5	107.7	116 0	124.3	132.6
40	8.50	17.00	25.50	34.00	42 50	51.00	59.50	68.00	76.50	85.00	93.5	102.0	110.5	119.0	127.5	136.0
41	8.71	17.43	26.14	34.85	43.56	52.28	60.99	69.70	78.41	87.13	95.8	104.6	113.3	122.0	130.7	139.4
42	8.93	17.85	26.78	35.70	44.63	53.55	62.48	71.40	80.33	89.25	98.2	107.1	116.0	125.0	133.9	142.8
43	9.14	18.28	27.41	36.55	45.69	54.83	63.96	73.10	82.24	91.38	100.5	109.7	118.8	127.9	137.1	146.2
44	9.35	18.70	28.05	37.40	46.75	56.10	65.45	74.80	84.15	93.50	102.9	112.2	121.6	130 9	140.3	149.6
45	9.56	19.13	28.69	38.25	47.81	57.38	66.94	76.50	86.06	95.63	105.2	114.8	124.3	133.9	143.4	153.0
46	9.78	19.55	29.33	39.10	48.88	58.65	68.43	78.20	87.98	97.75	107.5	117.3	127.1	136.9	146.6	156.4
47	9.99	19.98	29.96	39.95	49.94	59.93	69.91	79 90	89.89	99.88	109.9	119.9	129.8	139.8	149.8	159.8
48	10.20	20.40	30.60	40.80	51.00	61.20	71.40	81.60	91 80	102.0	112.2	122.4	132.6	142 8	153.0	163.2

TABLE 2.—*Continued.*

Weights of Steel Bars and Plates.

[redacted]

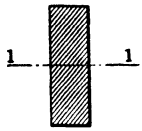
Pounds per Lineal Foot.

Width, Inches.	1/16	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
	Thickness, Inches.															
49	10.4	20.8	31.2	41.7	52.1	62.5	72.9	83.3	93.7	104.1	114.5	125.0	135.4	145.8	156.2	166.6
50	10.6	21.3	31.9	42.5	53.1	63.8	74.4	85.0	95.6	106.3	116.9	127.5	138.1	148.8	159.4	170.0
51	10.8	21.7	32.5	43.4	54.2	65.0	75.9	86.7	97.5	108.4	119.2	130.1	140.9	151.7	162.6	173.4
52	11.1	22.1	33.2	44.2	55.3	66.3	77.4	88.4	99.5	110.5	121.6	132.6	143.7	154.7	165.8	176.8
53	11.3	22.5	33.8	45.1	56.3	67.6	78.8	90.1	101.4	112.6	123.9	135.2	146.4	157.7	168.9	180.2
54	11.5	23.0	34.4	45.9	57.4	68.9	80.3	91.8	103.3	114.8	126.2	137.7	149.2	160.7	172.1	183.6
55	11.7	23.4	35.1	46.8	58.4	70.1	81.8	93.5	105.2	116.9	128.6	140.3	151.9	163.6	175.3	187.0
56	11.9	23.8	35.7	47.6	59.5	71.4	83.3	95.2	107.1	119.0	130.9	142.8	154.7	166.6	178.5	190.4
57	12.1	24.2	36.3	48.5	60.6	72.7	84.8	96.9	109.0	121.1	133.2	145.4	157.5	169.6	181.7	193.8
58	12.3	24.7	37.0	49.3	61.6	74.0	86.3	98.6	110.9	123.3	135.6	147.9	160.2	172.6	184.9	197.2
59	12.5	25.1	37.6	50.2	62.7	75.2	87.8	100.3	112.8	125.4	137.9	150.5	163.0	175.5	188.1	200.6
60	12.8	25.5	38.3	51.0	63.8	76.5	89.3	102.0	114.8	127.5	140.3	153.0	165.8	178.5	191.3	204.0
61	13.0	25.9	38.9	51.9	64.8	77.8	90.7	103.7	116.7	129.6	142.6	155.6	168.5	181.5	194.4	207.4
62	13.2	26.4	39.5	52.7	65.9	79.1	92.2	105.4	118.6	131.8	144.9	158.1	171.3	184.5	197.6	210.8
63	13.4	26.8	40.2	53.6	66.9	80.3	93.7	107.1	120.5	133.9	147.3	160.7	174.0	187.4	200.8	214.2
64	13.6	27.2	40.8	54.4	68.0	81.6	95.2	108.8	122.4	136.0	149.6	163.2	176.8	190.4	204.0	217.6
65	13.8	27.6	41.4	55.3	69.1	82.9	96.7	110.5	124.3	138.1	151.9	165.8	179.6	193.4	207.2	221.0
66	14.0	28.1	42.1	56.1	70.1	84.2	98.2	112.2	126.2	140.3	154.3	168.3	182.3	196.4	210.4	224.4
67	14.2	28.5	42.7	57.0	71.2	85.4	99.7	113.9	128.1	142.4	156.6	170.9	185.1	199.3	213.6	227.8
68	14.5	28.9	43.4	57.8	72.3	86.7	101.2	115.6	130.1	144.5	159.0	173.4	187.9	202.3	216.8	231.2
69	14.7	29.3	44.0	58.7	73.3	88.0	102.6	117.3	132.0	146.6	161.3	176.0	190.6	205.3	219.9	234.6
70	14.9	29.8	44.6	59.5	74.4	89.3	104.1	119.0	133.9	148.8	163.6	178.5	193.4	208.3	223.1	238.0
71	15.1	30.2	45.3	60.4	75.4	90.5	105.6	120.7	135.8	150.9	166.0	181.1	196.1	211.2	226.3	241.4
72	15.3	30.6	45.9	61.2	76.5	91.8	107.1	122.4	137.7	153.0	168.3	183.6	198.9	214.2	229.5	244.8
73	15.5	31.0	46.5	62.1	77.6	93.1	108.6	124.1	139.6	155.1	170.6	186.2	201.7	217.2	232.7	248.2
74	15.7	31.5	47.2	62.9	78.6	94.4	110.1	125.8	141.5	157.3	173.0	188.7	204.4	220.2	235.9	251.6
75	15.9	31.9	47.8	63.8	79.7	95.6	111.6	127.5	143.4	159.4	175.3	191.3	207.2	223.1	239.1	255.0
76	16.2	32.3	48.5	64.6	80.8	96.9	113.1	129.2	145.4	161.5	177.7	193.8	210.0	226.1	242.3	258.4
77	16.4	32.7	49.1	65.5	81.8	98.2	114.5	130.9	147.3	163.6	180.0	196.4	212.7	229.1	245.4	261.8
78	16.6	33.2	49.7	66.3	82.9	99.5	116.0	132.6	149.2	165.8	182.3	198.9	215.5	232.1	248.6	265.2
79	16.8	33.6	50.4	67.2	83.9	100.7	117.5	134.3	151.1	167.9	184.7	201.5	218.2	235.0	251.8	268.6
80	17.0	34.0	51.0	68.0	85.0	102.0	119.0	136.0	153.0	170.0	187.0	204.0	221.0	238.0	255.0	272.0
81	17.2	34.4	51.6	68.9	86.1	103.3	120.5	137.7	154.9	172.1	189.3	206.6	223.8	241.0	258.2	275.4
82	17.4	34.9	52.3	69.7	87.1	104.6	122.0	139.4	156.8	174.3	191.7	209.1	226.5	244.0	261.4	278.8
83	17.6	35.3	52.9	70.6	88.2	105.8	123.5	141.1	158.7	176.4	194.0	211.7	229.3	246.9	264.6	282.2
84	17.9	35.7	53.6	71.4	89.3	107.1	125.0	142.8	160.7	178.5	196.4	214.2	232.1	249.9	267.8	285.6
85	18.1	36.1	54.2	72.3	90.3	108.4	126.4	144.5	162.6	180.6	198.7	216.8	234.8	252.9	270.9	289.0
86	18.3	36.6	54.8	73.1	91.4	109.7	127.9	146.2	164.5	182.8	201.0	219.3	237.6	255.9	274.1	292.4
87	18.5	37.0	55.5	74.0	92.4	110.9	129.4	147.9	166.4	184.9	203.4	221.9	240.3	258.8	277.3	295.8
88	18.7	37.4	56.1	74.8	93.5	112.2	130.9	149.6	168.3	187.0	205.7	224.4	243.1	261.8	280.5	299.2
89	18.9	37.8	56.7	75.7	94.6	113.5	132.4	151.3	170.2	189.1	208.0	227.0	245.9	264.8	283.7	302.6
90	19.1	38.3	57.4	76.5	95.6	114.8	133.9	153.0	172.1	191.3	210.4	229.5	248.6	267.8	286.9	306.0
91	19.3	38.7	58.0	77.4	96.7	116.0	135.4	154.7	174.0	193.4	212.7	232.1	251.4	270.7	290.1	309.4
92	19.6	39.1	58.7	78.2	97.8	117.3	136.9	156.4	176.0	195.5	215.1	234.6	254.2	273.7	293.3	312.8
93	19.8	39.5	59.3	79.1	98.8	118.6	138.3	158.1	177.9	197.6	217.4	237.2	256.9	276.7	296.4	316.2
94	20.0	40.0	59.9	79.9	99.9	119.9	139.8	159.8	179.8	199.8	219.7	239.7	259.7	279.7	299.6	319.6
95	20.2	40.4	60.6	80.8	100.9	121.1	141.3	161.5	181.7	201.9	222.1	242.3	262.4	282.6	302.8	323.0
96	20.4	40.8	61.2	81.6	102.0	122.4	142.8	163.2	183.6	204.0	224.4	244.8	265.2	285.6	306.0	326.4
97	20.6	41.2	61.8	82.5	103.1	123.7	144.3	164.9	185.5	206.1	226.7	247.4	268.0	288.6	309.2	329.8
98	20.8	41.7	62.5	83.3	104.1	125.0	145.8	166.6	187.4	208.3	229.1	249.9	270.7	291.6	312.4	333.2
99	21.0	42.1	63.1	84.2	105.2	126.2	147.3	168.3	189.3	210.4	231.4	252.5	273.5	294.5	315.6	336.6
100	21.3	42.5	63.8	85.0	106.3	127.5	148.8	170.0	191.3	212.5	233.8	255.0	276.3	297.5	318.8	340.0

TABLE 3.

Moments of Inertia of Plates, Axis 1–1.

Moments of Inertia of One Plate.



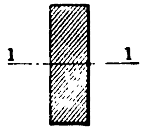
About Axis 1–1.

Width in Inches.	Thickness of Plate in Inches.												
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
5	2.6	3.3	3.9	4.6	5.2	5.9	6.5	7.2	7.8	8.5	9.1	9.8	10.4
6	4.5	5.6	6.8	7.9	9.0	10.1	11.3	12.4	13.5	14.6	15.8	16.9	18.0
7	7.1	8.9	10.7	12.5	14.3	16.1	17.9	19.6	21.4	23.2	25.0	26.8	28.6
8	10.7	13.3	16.0	18.7	21.3	24.0	26.7	29.3	32.0	34.7	37.3	40.0	42.7
9	15.2	19.0	22.8	26.6	30.4	34.2	38.0	41.8	45.6	49.4	53.2	57.0	60.7
10	20.8	26.0	31.3	36.5	41.7	46.9	52.1	57.3	62.5	67.7	72.9	78.1	83.3
11	27.7	34.7	41.6	48.5	55.5	62.4	69.3	76.3	83.2	90.1	97.0	104.0	110.9
12	36.0	45.0	54.0	63.0	72.0	81.0	90.0	99.0	108.0	117.0	126.0	135.0	144.0
13	45.8	57.2	68.7	80.1	91.5	103.0	114.4	125.9	137.3	148.8	160.2	171.6	183.1
14	57.2	71.5	85.8	100.0	114.3	128.6	142.9	157.2	171.5	185.8	200.1	214.4	228.7
15	70.3	87.9	105.5	123.0	140.6	158.2	175.8	193.4	210.9	228.5	246.1	263.7	281.2
16	85.3	106.7	128.0	149.3	170.7	192.0	213.3	234.7	256.0	277.3	298.7	320.0	341.3
17	102.4	127.9	153.5	179.1	204.7	230.3	255.9	281.5	307.1	332.7	358.2	383.8	409.4
18	121.5	151.9	182.3	212.6	243.0	273.4	303.8	334.1	364.5	394.9	425.3	455.6	486.0
19	142.9	178.6	214.3	250.1	285.8	321.5	357.2	393.0	428.7	464.4	500.1	535.9	571.6
20	166.7	208.3	250.0	291.7	333.3	375.0	416.7	458.3	500.0	541.7	583.3	625.0	666.7
21	192.9	241.2	289.4	337.6	385.9	434.1	482.3	530.6	578.8	627.0	675.3	723.5	771.7
22	221.8	277.3	332.7	388.2	443.7	499.1	554.6	610.0	665.5	721.0	776.4	831.9	887.3
23	253.5	316.9	380.2	443.6	507.0	570.3	633.7	697.1	760.4	823.8	887.2	950.6	1013.9
24	288.0	360.0	432.0	504.0	576.0	648.0	720.0	792.0	864.0	936.0	1008.0	1080.0	1152.0
25	325.5	406.9	488.3	569.7	651.0	732.4	813.8	895.2	976.6	1057.9	1139.3	1220.7	1302.1
26	366.2	457.7	549.3	640.8	732.3	823.9	915.4	1007.0	1098.5	1190.0	1281.6	1373.1	1464.7
27	410.1	512.6	615.1	717.6	820.1	922.6	1025.2	1127.7	1230.2	1332.7	1435.2	1537.7	1640.3
28	457.3	571.7	686.0	800.3	914.7	1029.0	1143.3	1257.7	1372.0	1486.3	1600.7	1715.0	1829.3
29	508.1	635.1	762.2	889.2	1016.2	1143.2	1270.3	1397.3	1524.3	1651.3	1778.4	1905.4	2032.4
30	562.5	703.1	843.8	984.4	1125.0	1265.6	1406.3	1546.9	1687.5	1828.1	1968.8	2109.4	2250.0
31	620.6	775.8	931.0	1086.1	1241.3	1396.5	1551.6	1706.8	1861.9	2017.1	2172.3	2327.4	2482.6
32	682.7	853.3	1024.0	1194.7	1365.3	1536.0	1706.7	1877.3	2048.0	2218.7	2389.3	2560.0	2730.7
33	748.7	935.9	1123.0	1310.2	1497.4	1684.5	1871.7	2058.9	2246.1	2433.2	2620.4	2807.6	2994.8
34	818.8	1023.5	1228.2	1433.0	1637.7	1842.4	2047.1	2251.8	2456.5	2661.2	2865.9	3070.6	3275.3
35	893.2	1116.5	1339.8	1563.2	1786.5	2009.8	2233.1	2456.4	2679.7	2903.0	3126.3	3349.6	3572.9
36	972.0	1215.0	1458.0	1701.0	1944.0	2187.0	2430.0	2673.0	2916.0	3159.0	3402.0	3645.0	3888.0
37	1055.3	1319.1	1582.9	1846.7	2110.5	2374.4	2638.2	2902.0	3165.8	3429.6	3693.4	3957.3	4221.1
38	1143.2	1429.0	1714.7	2000.5	2286.3	2572.1	2857.9	3143.7	3429.5	3715.3	4001.1	4286.9	4572.7
39	1235.8	1544.8	1853.7	2162.7	2471.6	2780.6	3089.5	3398.5	3707.4	4016.4	4325.3	4634.3	4943.2
40	1333.3	1666.7	2000.0	2333.3	2666.7	3000.0	3333.3	3666.7	4000.0	4333.3	4666.7	5000.0	5333.3
41	1435.9	1794.8	2153.8	2512.7	2871.7	3230.7	3589.6	3948.6	4307.6	4666.5	5025.5	5384.5	5743.4
42	1543.5	1929.4	2315.3	2701.1	3087.0	3472.9	3858.8	4244.6	4630.5	5016.4	5402.3	5788.2	6174.0
43	1656.4	2070.5	2484.6	2898.7	3312.8	3726.9	4141.0	4555.0	4969.2	5383.3	5797.4	6211.5	6625.6
44	1774.7	2218.3	2662.0	3105.7	3549.3	3993.0	4436.7	4880.3	5324.0	5767.7	6211.3	6655.0	7098.7

TABLE 3.—*Continued.*

MOMENTS OF INERTIA OF PLATES, AXIS 1–1.

Moments of Inertia of One Plate.



About Axis 1–1.

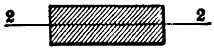
Thickness of Plate in Inches.

Width in Inches	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
45	1898	2373	2848	3322	3797	4271	4746	5221	5695	6170	6645	7119	7594
46	2028	2535	3042	3549	4056	4563	5070	5577	6083	6590	7097	7604	8111
47	2163	2704	3244	3785	4326	4867	5407	5948	6489	7030	7570	8111	8652
48	2304	2880	3456	4032	4608	5184	5760	6336	6912	7488	8064	8640	9216
49	2451	3064	3677	4289	4902	5515	6128	6740	7353	7966	8579	9191	9804
50	2604	3255	3906	4557	5208	5859	6510	7161	7812	8464	9115	9766	10417
52	2929	3662	4394	5126	5859	6591	7323	8056	8788	9520	10253	10985	11717
54	3280	4101	4921	5741	6561	7381	8201	9021	9841	10662	11482	12302	13122
56	3659	4573	5488	6403	7317	8232	9147	10061	10976	11891	12805	13720	14635
58	4065	5081	6097	7113	8130	9146	10162	11178	12194	13211	14227	15243	16259
60	4500	5625	6750	7875	9000	10125	11250	12375	13500	14625	15750	16875	18000
62	4965	6206	7448	8689	9930	11172	12413	13654	14895	16137	17378	18619	19861
64	5461	6827	8192	9557	10923	12288	13653	15019	16384	17749	19115	20480	21845
66	5989	7487	8984	10482	11979	13476	14974	16471	17968	19466	20963	22461	23958
68	6551	8188	9826	11464	13101	14739	16377	18014	19652	21290	22927	24565	26203
70	7145	8932	10719	12505	14292	16078	17865	19651	21437	23224	25010	26797	28583
72	7776	9720	11664	13608	15552	17496	19440	21384	23328	25272	27216	29160	31104
74	8442	10553	12663	14774	16884	18995	21105	23216	25326	27437	29548	31658	33769
76	9145	11432	13718	16004	18291	20577	22863	25150	27436	29722	32009	34295	36581
78	9886	12358	14830	17301	19773	22245	24716	27188	29659	32131	34603	37074	39546
80	10667	13333	16000	18667	21333	24000	26667	29333	32000	34667	37333	40000	42667
82	11487	14359	17230	20102	22974	25845	28717	31589	34460	37332	40204	43076	45947
84	12348	15435	18522	21609	24696	27783	30870	33957	37044	40131	43218	46305	49392
86	13251	16564	19877	23190	26502	29815	33128	36441	39753	43066	46379	49692	53005
88	14197	17747	21296	24845	28395	31944	35493	39043	42592	46141	49691	53240	56789
90	15187	18984	22781	26578	30375	34172	37969	41766	45562	49359	53156	56953	60750
92	16223	20278	24334	28390	32445	36501	40557	44612	48668	52724	56779	60835	64891
94	17304	21630	25956	30282	34608	38934	43260	47586	51911	56237	60563	64889	69215
96	18432	23040	27648	32256	36864	41472	46080	50688	55296	59904	64512	69120	73728
98	19608	24510	29412	34314	39216	44118	49020	53922	58824	63727	68629	73531	78433
100	20833	26042	31250	36458	41667	46875	52083	57292	62500	67708	72917	78125	83333
102	22108	27636	33163	38690	44217	49744	55271	60798	66325	71853	77380	82907	88434
104	23435	29293	35152	41011	46869	52728	58587	64445	70304	76163	82021	87880	93739
106	24813	31016	37219	43422	49626	55829	62032	68235	74438	80642	86845	93048	99251
108	26244	32805	39366	45927	52488	59049	65610	72171	78732	85293	91854	98415	104976
110	27729	34661	41594	48526	55458	62391	69323	76255	83187	90120	97052	103984	110917
112	29269	36587	43904	51221	58539	65856	73173	80491	87808	95125	102443	109760	117077
114	30865	38582	46298	54015	61731	69447	77164	84880	92596	100313	108029	115746	123462
116	32519	40648	48778	56908	65037	73167	81297	89426	97556	105686	113815	121945	130075
118	34230	42787	51345	59902	68460	77017	85575	94132	102689	111247	119804	128362	136919
120	36000	45000	54000	63000	72000	81000	90000	99000	108000	117000	126000	135000	144000

TABLE 4.

Moments of Inertia of Plates, Axis 2–2.

Moments of Inertia of One Plate.



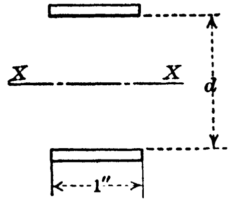
About Axis 2–2.

Width in Inches.	Thickness of Plate in Inches.												
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
5	.01	.01	.02	.03	.05	.07	.10	.14	.18	.22	.28	.34	.42
6	.01	.02	.03	.04	.06	.09	.12	.16	.21	.27	.33	.41	.50
7	.01	.02	.03	.05	.07	.10	.14	.19	.25	.31	.39	.48	.58
8	.01	.02	.04	.06	.08	12	.16	.22	.28	.36	.45	.55	.67
9	.01	.02	.04	.06	.09	.13	.18	.24	.32	.40	.50	.62	.75
10	.01	.03	.04	.07	.10	.15	.20	.27	.35	.45	.56	.69	.83
11	.01	.03	.05	.08	.11	.16	.22	.30	.39	.49	.61	.76	.92
12	.02	.03	.05	.08	.13	.18	.24	.33	.42	.54	.67	.82	1.00
13	.02	.03	.06	.09	.14	.19	.26	.35	.46	.58	.73	.89	1.08
14	.02	.04	.06	.10	.15	.21	.28	.38	.49	.63	.78	.96	1.17
15	.02	.04	.07	.10	.16	.22	.31	.41	.53	.67	.84	1.03	1.25
16	.02	.04	.07	.11	.17	.24	.33	.43	.56	.72	.89	1.10	1.33
17	02	.04	.07	.12	.18	.25	.35	.46	.60	.76	.95	1.17	1.42
18	.02	.05	.08	.13	.19	.27	.37	.49	.63	.80	1.00	1.24	1.50
19	.02	.05	.08	.13	.20	.28	.39	.51	.67	.85	1.06	1.30	1.58
20	.03	.05	.09	.14	.21	.30	.41	.54	.70	.89	1.12	1.37	1.67
21	.03	.05	.09	.15	.22	.31	.43	.57	.74	.94	1.17	1.44	1.75
22	.03	.06	.10	.15	.23	.33	.45	.60	.77	.98	1.23	1.51	1.83
23	.03	.06	.10	.16	.24	.34	.47	.62	.81	1.03	1.28	1.58	1.92
24	.03	.06	.11	.17	.25	.36	.49	.65	.84	1.07	1.34	1.65	2.00
25	.03	.06	.11	.17	.26	.37	.51	.68	.88	1.12	1.40	1.72	2.08
26	.03	.07	.11	.18	.27	.39	.53	.70	.91	1.16	1.45	1.79	2.17
27	.04	.07	.12	.19	.28	.40	.55	.73	.95	1.21	1.51	1.85	2.25
28	.04	.07	.12	.20	.29	.42	.57	.76	.98	1.25	1.56	1.92	2.33
29	.04	.07	.13	.20	.30	.43	.59	.79	1.02	1.30	1.62	1.99	2.42
30	.04	.08	.13	.21	.31	.44	.61	.81	1.05	1.34	1.67	2.06	2.50
32	.04	.08	.14	.22	.33	.47	.65	.87	1.12	1.43	1.79	2.20	2.67
34	.04	.09	.15	.24	.35	.50	.69	.92	1.20	1.52	1.90	2.33	2.83
36	.05	.09	.16	.25	.38	.53	.73	.98	1.27	1.61	2.01	2.47	3.00
38	.05	.10	.17	.27	.40	.56	.77	1.03	1.34	1.70	2.12	2.61	3.17
40	.05	.10	.18	.28	.42	.59	.81	1.08	1.41	1.79	2.23	2.75	3.33
42	.05	.11	.18	.29	.44	.62	.85	1.14	1.48	1.88	2.34	2.88	3.50
44	.06	.11	.19	.31	.46	.65	.90	1.19	1.55	1.97	2.46	3.02	3.67
46	.06	.12	.20	.32	.48	.68	.94	1.25	1.62	2.06	2.57	3.16	3.83
48	.06	.12	.21	.33	.50	.71	.98	1.30	1.69	2.15	2.68	3.30	4.00
50	.07	.13	.22	.35	.52	.74	1.02	1.35	1.76	2.23	2.79	3.43	4.17
52	.07	.13	.23	.36	.54	.77	1.05	1.41	1.82	2.32	2.90	3.57	4.33
54	.07	.14	.24	.38	.56	.80	1.10	1.46	1.90	2.41	3.01	3.71	4.50
56	.08	.14	.25	.39	.58	.83	1 14	1.52	1.96	2.50	3.13	3.85	4.67
58	.08	.15	.25	.41	.60	.86	1.18	1.57	2.04	2.59	3.24	3.98	4.83
60	.08	.15	.26	.42	.63	.89	1.22	1.63	2.11	2.68	3.35	4.12	5.00

TABLE 5.

Moments of Inertia of Two Plates One Inch Wide, Axis X–X.

Moments of Inertia of Two Plates One Inch Wide, Axis X–X.



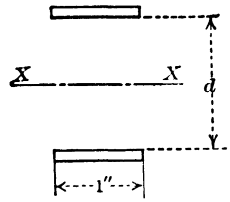
For Distances Measured from Inside to Inside

d Ins.	Thickness of Plate in Inches.													
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	$\frac{1}{8}$
5	3.4	4.4	5.4	6.5	7.6	8.7	9.9	11.2	12.5	13.8	15.2	16.6	18.2	1.6
5¼	3.8	4.8	5.9	7.1	8.3	9.5	10.8	12.2	13.6	15.0	16.5	18.1	19.7	1.8
5½	4.1	5.3	6.5	7.7	9.0	10.4	11.8	13.2	14.7	16.3	17.9	19.6	21.3	2.0
5¾	4.5	5.7	7.0	8.4	9.8	11.2	12.7	14.3	15.9	17.6	19.3	21.1	22.9	2.2
6	4.9	6.2	7.6	9.1	10.6	12.1	13.8	15.4	17.2	18.9	20.7	22.7	24.7	2.3
6¼	5.3	6.7	8.2	9.8	11.4	13.1	14.8	16.6	18.5	20.4	22.3	24.4	26.5	2.5
6½	5.7	7.3	8.9	10.5	12.3	14.1	15.9	17.8	19.8	21.8	23.9	26.1	28.3	2.7
6¾	6.1	7.8	9.5	11.3	13.2	15.1	17.0	19.1	21.2	23.3	25.5	27.8	30.2	3.0
7	6.6	8.4	10.2	12.1	14.1	16.1	18.2	20.4	22.6	24.9	27.2	29.7	32.2	3.2
7¼	7.0	8.9	10.9	12.9	15.0	17.2	19.4	21.7	24.1	26.5	29.0	31.6	34.2	3.4
7½	7.5	9.5	11.6	13.8	16.0	18.3	20.7	23.1	25.6	28.2	30.8	33.5	36.3	3.6
7¾	8.0	10.2	12.4	14.7	17.0	19.5	22.0	24.5	27.2	29.9	32.7	35.5	38.4	3.9
8	8.5	10.8	13.2	15.6	18.1	20.6	23.3	26.0	28.8	31.6	34.6	37.6	40.7	4.1
8¼	9.0	11.5	14.0	16.5	19.2	21.9	24.7	27.5	30.5	33.5	36.5	39.7	43.0	4.4
8½	9.6	12.1	14.8	17.5	20.3	23.1	26.1	29.1	32.2	35.3	38.6	41.9	45.3	4.6
8¾	10.1	12.8	15.6	18.5	21.4	24.4	27.5	30.7	33.9	37.2	40.6	44.1	47.7	4.9
9	10.7	13.6	16.5	19.5	22.6	25.7	29.0	32.3	35.7	39.2	42.8	46.4	50.2	5.2
9¼	11.3	14.3	17.4	20.5	23.8	27.1	30.5	34.0	37.6	41.2	45.0	48.8	52.7	5.5
9½	11.9	15.0	18.3	21.6	25.0	28.5	32.1	35.7	39.5	43.3	47.2	51.2	55.3	5.8
9¾	12.5	15.8	19.2	22.7	26.3	29.9	33.7	37.5	41.4	45.4	49.5	53.7	57.9	6.1
10	13.1	16.6	20.2	23.8	27.6	31.4	35.3	39.3	43.4	47.6	51.9	56.2	60.7	6.4
10¼	13.8	17.4	21.2	25.0	28.9	32.9	37.0	41.2	45.5	49.8	54.3	58.8	63.5	6.7
10½	14.5	18.3	22.2	26.2	30.3	34.5	38.7	43.1	47.5	52.1	56.7	61.5	66.3	7.1
10¾	15.1	19.1	23.2	27.4	31.7	36.0	40.5	45.0	49.7	54.4	59.2	64.2	69.2	7.4
11	15.8	20.0	24.3	28.6	33.1	37.6	42.3	47.0	51.9	56.8	61.8	66.9	72.2	7.7
11¼	16.5	20.9	25.4	29.9	34.5	39.3	44.1	49.0	54.1	59.2	64.4	69.8	75.2	8.1
11½	17.3	21.8	26.5	31.2	36.0	40.9	46.0	51.1	56.4	61.7	67.1	72.7	78.3	8.4
11¾	18.0	22.7	27.6	32.5	37.5	42.7	47.9	53.2	58.7	64.2	69.8	75.6	81.4	8.8
12	18.8	23.7	28.7	33.9	39.1	44.2	49.8	55.4	61.0	66.8	72.6	76.8	84.7	9.2
12¼	19.5	24.7	29.9	35.2	40.7	46.2	51.8	57.6	63.5	69.4	75.5	81.7	88.0	9.6
12½	20.3	25.7	31.1	36.6	42.3	48.0	53.9	59.8	65.9	72.1	78.4	84.8	91.3	10.0
12¾	21.1	26.7	32.3	38.1	43.9	49.9	55.9	62.1	68.4	74.8	81.3	88.0	94.7	10.4
13	21.9	27.7	33.6	39.5	45.6	51.8	58.1	64.5	71.0	77.6	84.3	91.2	98.2	10.8
13¼	22.8	28.8	34.8	41.0	47.3	53.7	60.2	66.8	73.6	80.4	87.4	94.5	101.7	11.2
13½	23.6	29.8	36.1	42.5	49.0	55.6	62.4	69.3	76.2	83.3	90.5	97.8	105.3	11.6
13¾	24.5	30.9	37.4	44.0	50.8	57.6	64.6	71.7	78.9	86.2	93.7	101.3	108.9	12.0
14	25.4	32.0	38.8	45.6	52.6	59.7	66.9	74.2	81.7	89.2	96.9	104.7	112.7	12.5
14¼	26.3	33.1	40.1	47.2	54.4	61.7	69.2	76.8	84.5	92.3	100.2	108.3	116.5	12.9
14½	27.2	34.3	41.5	48.8	56.3	63.8	71.5	79.4	87.3	95.3	103.5	111.9	120.3	13.4
14¾	28.1	35.5	42.9	50.5	58.2	66.0	73.9	82.0	90.2	98.4	106.9	115.5	124.2	13.8
15	29.1	36.7	44.3	52.1	60.1	68.1	76.3	84.7	93.1	101.7	110.4	119.2	128.2	14.3
15¼	30.0	37.9	45.8	53.9	62.0	70.4	78.8	87.4	96.1	104.9	113.9	123.0	132.2	14.8
15½	31.0	39.1	47.3	55.6	64.0	72.6	81.3	90.1	99.1	108.2	117.4	126.8	136.3	15.3
15¾	32.0	40.3	48.7	57.3	66.0	74.9	83.8	92.9	102.2	111.5	121.0	130.7	140.4	15.7

For Moment of Inertia, deducting for rivet holes, multiply tabular value by net width.

Moments of Inertia of Two Plates One Inch Wide, Axis X–X.

Moments of Inertia of Two Plates One Inch Wide, Axis X-X.



For Distances Measured from Inside to Inside

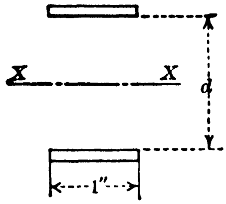
d Ins.	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1/8
	Thickness of Plate in Inches.													
16	33.0	41.6	50.2	59.1	68.1	77.2	86.4	95.8	105.3	114.9	124.7	134.6	144.7	16.2
16¼	34.0	42.9	51.8	60.9	70.2	79.5	89.0	98.7	108.5	118.4	128.4	138.6	149.0	16.8
16½	35.1	44.2	53.4	62.8	72.3	81.9	91.7	101.6	111.7	121.9	132.2	142.7	153.3	17.3
16¾	36.1	45.5	55.0	64.6	74.4	84.3	94.4	104.6	114.9	125.4	136.0	146.8	157.7	17.8
18¼	42.8	53.9	65.1	76.4	87.9	99.6	111.4	123.3	135.5	147.7	160.1	172.7	185.5	21.1
18½	43.9	55.3	66.8	78.5	90.3	102.2	114.3	126.6	139.0	151.6	164.3	177.2	190.3	21.7
20¼	52.5	66.1	79.8	93.6	107.7	121.9	136.2	150.8	165.5	180.3	195.4	210.6	226.0	26.0
20½	53.8	67.7	81.7	95.9	110.3	124.8	139.5	154.4	169.4	184.6	200.0	215.6	231.3	26.6
22¼	63.3	79.6	96.0	112.6	129.4	146.4	163.6	180.9	198.5	216.2	234.1	252.2	270.5	31.3
22½	64.7	81.3	98.1	115.1	132.3	149.6	167.2	184.9	202.8	220.9	239.2	257.6	276.3	32.0
24¼	75.0	94.3	113.7	133.3	153.2	173.2	193.4	213.8	234.5	255.3	276.3	297.5	319.0	37.1
24½	76.6	96.2	116.0	136.0	156.3	176.7	197.3	218.1	239.2	260.4	281.8	303.5	325.3	37.9
26¼	87.8	110.3	132.9	155.8	178.9	202.2	225.8	249.5	273.5	297.6	322.0	346.6	371.5	43.5
26½	89.4	112.3	135.4	158.7	182.3	206.0	230.0	254.1	278.5	303.1	328.0	353.0	378.3	44.3
28¼	101.5	127.5	153.7	180.0	206.7	233.5	260.6	287.9	315.5	343.2	371.2	399.5	428.0	50.3
28½	103.3	129.7	156.3	183.2	210.3	237.6	265.1	292.9	320.9	349.2	377.6	406.3	435.3	51.2
30¼	116.3	146.0	175.9	206.0	236.4	267.1	297.9	329.1	360.5	392.1	424.0	456.1	488.5	57.7
30½	118.2	148.4	178.7	209.4	240.3	271.4	302.8	334.4	366.3	398.4	430.8	463.4	496.3	58.6
32¼	132.0	165.7	199.6	233.8	268.2	302.8	337.8	373.0	408.5	444.2	480.2	516.4	553.0	65.5
32½	134.1	168.2	202.7	237.3	272.3	307.5	342.9	378.7	414.7	450.9	487.4	524.2	561.3	66.5
34¼	148.8	186.7	224.0	263.2	301.9	340.9	380.1	419.6	459.5	499.5	539.9	580.5	621.5	73.9
34½	150.9	189.4	228.1	267.0	306.3	345.8	385.6	425.7	466.0	506.7	547.6	588.8	630.3	74.9
36¼	166.5	208.9	251.5	294.5	337.7	381.2	425.0	469.1	513.5	558.1	603.1	648.3	694.0	82.7
36½	168.8	211.7	255.0	298.5	342.3	386.4	430.7	475.4	520.4	565.7	611.2	657.1	703.3	83.8
38¼	185.3	232.4	279.7	327.4	375.4	423.7	472.3	521.2	570.5	620.0	669.8	720.0	770.5	92.0
38½	187.7	235.4	283.4	331.7	380.3	429.2	478.4	527.9	577.8	627.9	678.4	729.2	780.3	93.2
40¼	205.0	257.1	309.5	362.2	415.2	468.5	522.2	576.1	630.5	685.1	740.1	795.3	851.0	101.9
40½	207.6	260.3	313.3	366.6	420.3	474.3	528.6	583.2	638.2	693.4	749.1	805.0	861.3	103.1
42¼	225.8	283.1	340.7	398.6	456.9	515.5	574.5	633.8	693.5	753.4	813.8	874.4	935.5	112.2
42½	228.4	286.4	344.7	403.3	462.3	521.6	581.2	641.2	701.5	762.2	823.2	884.6	946.3	113.6
44¼	247.5	310.3	373.4	436.9	500.7	564.8	629.4	694.2	759.5	825.0	891.0	957.3	1024.0	123.1
44½	250.3	313.8	377.6	441.7	506.3	571.1	636.4	702.0	767.9	834.2	900.9	967.9	1035.3	124.6
46¼	270.3	338.8	407.6	476.8	546.4	616.4	686.7	757.4	828.5	899.9	971.7	1043.9	1116.5	134.4
46½	273.2	342.4	412.0	481.9	552.3	623.0	694.0	765.5	837.3	909.5	982.0	1055.0	1128.3	135.9
48¼	294.0	368.5	443.4	518.6	594.2	670.2	746.5	823.3	900.5	978.0	1055.9	1134.3	1213.0	146.3
48½	297.1	372.3	447.9	523.9	600.3	677.0	754.2	831.7	909.7	988.0	1066.7	1145.8	1225.3	147.8
50¼	318.8	399.5	480.6	562.0	643.9	726.2	808.9	892.0	975.5	1059.4	1143.6	1228.4	1313.5	158.6
50½	321.9	403.4	485.3	567.6	650.3	733.4	816.8	900.7	985.0	1069.7	1154.8	1240.4	1326.3	160.2
52¼	344.5	431.7	519.3	607.3	695.7	784.5	873.7	963.4	1053.5	1144.0	1234.9	1326.2	1418.0	171.5
52½	347.8	435.8	524.2	613.0	702.3	791.9	882.0	972.5	1063.4	1154.7	1246.5	1338.7	1431.3	173.1
54¼	371.3	465.2	559.5	654.3	749.4	845.0	941.1	1037.5	1134.5	1231.8	1329.6	1427.8	1526.5	184.8
54½	374.7	469.4	564.6	660.2	756.3	852.7	949.7	1047.0	1144.8	1243.0	1341.7	1440.8	1540.3	186.5
56¼	399.0	499.9	601.2	703.0	805.2	907.8	1010.9	1114.5	1218.5	1322.9	1427.8	1533.2	1639.0	198.6
56½	402.6	504.3	606.5	709.2	812.3	915.8	1019.8	1124.3	1229.2	1334.5	1440.3	1546.6	1653.3	200.4

For Moment of Inertia, deducting for rivet holes, multiply tabular value by net width.

TABLE 5.—*Continued.*

Moments of Inertia of Two Plates One Inch Wide, Axis X-X.

Moments of Inertia of Two Plates One Inch Wide, Axis X-X.



For Distances Measured from Inside to Inside.

d Ins.	Thickness of Plate in Inches.													
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1/8
58¼	427.8	535.9	644.4	753.5	862.9	972.9	1083.3	1194.1	1305.5	1417.3	1529.5	1642.3	1755.5	213.0
58½	431.4	540.5	649.9	759.9	870.3	981.1	1092.5	1204.3	1316.5	1429.3	1542.5	1656.1	1770.3	214.8
60¼	427.5	573.1	689.2	805.7	922.7	1040.1	1158.1	1276.5	1395.5	1514.9	1634.7	1755.1	1876.0	227.8
60½	461.3	577.8	694.8	812.3	930.3	1048.7	1167.6	1287.0	1406.9	1527.3	1648.1	1769.5	1891.3	229.7
62¼	488.3	611.6	735.4	859.7	984.4	1109.7	1235.4	1361.7	1488.5	1615.7	1743.5	1871.7	2000 5	243.2
62½	492.2	616.5	741.2	866.5	992.3	1118.5	1245.3	1372.5	1500.3	1628.5	1757.3	1886.5	2016.3	245.1
64¼	520.0	651.3	783.1	915.4	1048.2	1181.5	1315.3	1449.6	1584.5	1719.8	1855.7	1992.1	2129.0	259.0
64½	524.1	656.4	789.1	922.4	1056.3	1190.6	1325.4	1460.8	1596.7	1733.0	1869.9	2007.4	2145.3	261.0
66¼	552.8	692.3	832.3	972.9	1113.9	1255.5	1397.6	1540.3	1683.5	1827.2	1971.4	2116.2	2261.5	275.4
66½	556.9	697.5	838.6	980.1	1122.3	1264.9	1408.1	1551.8	1696.0	1840.8	1986.1	2131.9	2278.3	277.4
68¼	586.5	734.5	883.0	1032.1	1181.7	1331.8	1482.5	1633.7	1785.5	1937.8	2090.6	2244.0	2398.0	292.2
68½	590.8	739.9	889.5	1039.6	1190.3	1341.5	1493.2	1 45.6	1798.4	1951.8	2105.7	2260.2	2415.3	294.3
70¼	621.3	778.0	935.3	1093.1	1251.4	1410.3	1569.8	1729.9	1890.5	2051.6	2213.3	2375.6	2538.5	309.6
70½	625.7	783.5	941.9	1100.8	1260.3	1420.3	1580.9	1742.1	1903.8	2066.1	2228.9	2392.3	2556.3	311.7
72¼	657.0	822.8	989.0	1155.8	1323.2	1491.1	1659.7	1828.8	1998.5	2168.7	2339.6	2511.0	2683.0	327.4
72½	661.6	828.4	995.8	1163.7	1332.3	1501.4	16/1.1	1841.3	2012.2	2183.6	2355.5	2528.1	2701.3	329.6
74½	698.4	874.5	1051.2	1228.4	1406.3	1584.7	1763.7	1943.3	2123.5	2304.3	2485.7	2667.7	2850.3	348.0
76½	736.3	921.9	1108.1	1294.9	1482.3	1670.3	1858.9	2048.1	2237.9	2428.3	2619.4	2811.0	3003.3	367.0
78½	775.2	970.5	1166.5	1363.1	1560.3	1758.1	1956.5	2155.6	2355.3	2555.6	2756.5	2958.1	3160.3	386.4
80½	815.1	1020.4	1226.4	1433.0	1640.3	1848.2	2056.7	2265.9	2475.7	2686.1	2897.2	3108.9	3321.3	406.3
82½	855.9	1071.6	1287.8	150[illegible].7	1722.3	1940.5	2159.3	2378.9	2599.0	2819.9	3041.3	3263.5	3486.3	426.7
84½	897.8	1123.9	1350.7	1578.1	1806.3	2035.0	2264.5	2494.6	2725.4	2956.9	3189.0	3421.8	3655.3	447.6
86½	940.7	1177.6	1415.1	1653.3	1892.3	2131.9	2372.1	2613.1	2854.8	3097.1	3340.1	3583.9	3828.3	469.0
88½	984.6	1232.5	1481.0	1730.3	1980.3	2230.9	2482.3	2734.4	2987.2	3240.6	3494.8	3749.7	4005.3	490.9
90½	1029.4	1288.6	1548.4	1809.0	2070.3	2332.3	2595.0	2858.4	3122.5	3387.4	3653.0	3919.3	4186.3	513.3
92½	1075.3	1346.0	1617.4	1889.4	2162.3	2435.8	2710.1	2985.2	3260.9	3537.4	3814.6	4092.6	4371.3	536.2
94½	1122.2	1404.6	1687.7	1971.6	2256.3	2541.6	2827.8	3114.7	3402.3	3690.7	3979.8	4269.7	4560.3	559.6
96½	1170.1	1464.5	1759.6	2055.6	2352.3	2649.7	2947.9	3246.9	3546.7	3847.2	4148.4	4450.5	4753.3	583.5
98½	1218.9	1525.6	1833.0	2141.3	2450.3	2760.0	3070.6	3381.9	3694.0	4006.9	4320.6	4635.0	4950.3	607.9
100½	1268.8	1588.0	1908.0	2228.7	2550.3	2872.6	3195.7	3519.7	3844.4	4169.9	4496.2	4823.4	5151.3	632.8
102½	1319.7	1651.6	1984.4	2317.9	2652.3	2987.4	3323.4	3660.2	3997.8	4336.2	4675.4	5015.4	5356.3	658.2
104½	1371.6	1716.5	2062.3	2408.8	2756.3	3104.5	3453.6	3803.5	4154.2	4505.7	4858.0	5211.3	5565.3	684.1
106½	1424.4	1782.7	2141.7	2501.5	2862.3	3223.8	3586.2	3949.5	4313.5	4678.5	5044.2	5410.8	5778.3	710.5
108½	1478.3	1850.0	2222.6	2596.0	2970.3	3345.4	3721.4	4098.2	4475.9	4854.5	5233.9	5614.1	5995.3	737.5
110½	1533.2	1918.7	2305.0	2692.2	3080.3	3469.2	3859.0	4249.7	4641.3	5033.7	5427.0	5821.2	6216.3	764.9
112½	1589.1	1988.6	2388.9	2790.1	3192.3	3595.3	3999.2	4404.0	4809.7	5216.2	5623.7	6032.0	6441.3	792.8
114½	1645.9	2059.7	2474.3	2889.8	3306.3	3723.6	4141.8	4561.0	4981.0	5402.0	5823.8	6246.6	6670.3	821.2
116½	1703.8	2132.1	2561.2	2991.3	3422.3	3854.2	4287.0	4720.8	5155.4	5591.0	6027.5	6464.9	6903.3	850.1
118½	1762.7	2205.7	2649.6	3094.5	3540.3	3987.0	4434.6	4883.3	5332.8	5783.3	6234.6	6687.0	7140.3	879.5
120½	1822.6	2280.6	2739.5	3199.4	3660.3	4122.1	4584.8	5048.5	5513.2	5978.8	6445.3	6912.8	7381.3	909.4

For Moment of Inertia, deducting for rivets, multiply tabular value by net width.

TABLE 6.

Weights and Areas of Square and Round Bars and Circumferences of Round Bars
One Cubic Foot of Steel Weighing 489.6 lb.

Thickness or Diameter in Inches.	Weight of ■ Bar One Ft. Long.	Weight of ● Bar One Ft. Long.	Area of ■ Bar in Sq. Inches.	Area of ● Bar in Sq. Inches.	Circumference of ○ Bar in Inches.	Thickness or Diameter in Inches.	Weight of ■ Bar One Ft Long.	Weight of ● Bar One Ft. Long.	Area of ■ Bar in Sq. Inches.	Area of ● Bar in Sq. Inches.	Circumference of ○ Bar in Inches.
0	......	......	......	......	......	3	30.60	24.03	9.0000	7.0686	9.424
1/16	.013	.010	.0039	.0031	.1963	1/16	31.89	25.04	9.3789	7.3662	9.621
1/8	.053	.042	.0156	.0123	.3927	1/8	33.20	26.08	9.7656	7.6699	9.817
3/16	.119	.094	.0352	.0276	.5890	3/16	34.55	27.13	10.160	7.9798	10.014
1/4	.212	.167	.0625	.0491	.7854	1/4	35.92	28.20	10.563	8.2958	10.210
5/16	.333	.261	.0977	.0767	.9817	5/16	37.31	29.30	10.973	8.6179	10.407
3/8	.478	.375	.1406	.1104	1.1781	3/8	38.73	30.42	11.391	8.9462	10.603
7/16	.651	.511	.1914	.1503	1.3744	7/16	40.18	31.56	11.816	9.2806	10.799
1/2	.850	.667	.2500	.1963	1.5708	1/2	41.65	32.71	12.250	9.6211	10.996
9/16	1.076	.845	.3164	.2485	1.7671	9/16	43.14	33.90	12.691	9.9678	11.192
5/8	1.328	1.043	.3906	.3068	1.9635	5/8	44.68	35.09	13.141	10.321	11.388
11/16	1.608	1.262	.4727	.3712	2.1598	11/16	46.24	36.31	13.598	10.680	11.585
3/4	1.913	1.502	.5625	.4418	2.3562	3/4	47.82	37.56	14.063	11.045	11.781
13/16	2.245	1.763	.6602	.5185	2.5525	13/16	49.42	38.81	14.535	11.416	11.977
7/8	2.603	2.044	.7656	.6013	2.7489	7/8	51.05	40.10	15.016	11.793	12.174
15/16	2.989	2.347	.8789	.6903	2.9452	15/16	52.71	41.40	15.504	12.177	12.370
1	3.400	2.670	1.0000	.7854	3.1416	4	54.40	42.73	16.000	12.566	12.566
1/16	3.838	3.014	1.1289	.8866	3.3379	1/16	56.11	44.07	16.504	12.962	12.763
1/8	4.303	3 379	1.2656	.9940	3 5343	1/8	57.85	45.44	17.016	13.364	12.959
3/16	4.795	3.766	1.4102	1.1075	3.7306	3/16	59.62	46.83	17.535	13.772	13.155
1/4	5.312	4.173	1.5625	1.2272	3.9270	1/4	61.41	48.24	18.063	14.186	13.352
5/16	5.857	4.600	1.7227	1.3530	4.1233	5/16	63.23	49.66	18.598	14.607	13.548
3/8	6.428	5.049	1.8906	1.4849	4.3197	3/8	65.08	51.11	19.141	15.033	13.744
7/16	7.026	5.518	2.0664	1.6230	4.5160	7/16	66.95	52.58	19.691	15.466	13.941
1/2	7.650	6.008	2.2500	1.7671	4.7124	1/2	68.85	54.07	20.250	15.904	14.137
9/16	8.301	6.520	2.4414	1.9175	4.9087	9/16	70.78	55.59	20.816	16.349	14.334
5/8	8.978	7.051	2.6406	2.0739	5.1051	5/8	72.73	57.12	21.391	16.800	14.530
11/16	9.682	7.604	2.8477	2.2365	5.3014	11/16	74.70	58.67	21.973	17.257	14.726
3/4	10.41	8.178	3.0625	2.4053	5.4978	3/4	76.71	60.25	22.563	17.721	14.923
13/16	11.17	8.773	3.2852	2.5802	5.6941	13/16	78.74	61.84	23.160	18.190	15.119
7/8	11.95	9.388	3.5156	2.7612	5.8905	7/8	80.81	63.46	23.766	18.665	15.315
15/16	12.76	10.02	3.7539	2.9483	6.0868	15/16	82.89	65.10	24.379	19.147	15.512
2	13.60	10.68	4.0000	3.1416	6.2832	5	85.00	66.76	25.000	19.635	15.708
1/16	14.46	11.36	4.2539	3.3410	6.4795	1/16	87.14	68.44	25.629	20.129	15.904
1/8	15.35	12.06	4.5156	3.5466	6.6759	1/8	89.30	70.14	26.266	20.629	16.101
3/16	16.27	12.78	4.7852	3.7583	6.8722	3/16	91.49	71.86	26.910	21.135	16.297
1/4	17.22	13.52	5.0625	3.9761	7.0686	1/4	93.72	73.60	27.563	21.648	16.493
5/16	18.19	14.28	5.3477	4.2000	7.2649	5/16	95.96	75.37	28.223	22.166	16.690
3/8	19.18	15.07	5.6406	4.4301	7.4613	3/8	98.23	77.15	28.891	22.691	16.886
7/16	20.20	15.86	5.9414	4.6664	7.6576	7/16	100.5	78.95	29.566	23.221	17.082
1/2	21.25	16.69	6.2500	4.9087	7.8540	1/2	102.8	80.77	30.250	23.758	17.279
9/16	22.33	17.53	6.5664	5.1572	8.0503	9/16	105 2	82.62	30.941	24.301	17.475
5/8	23.43	18.40	6.8906	5.4119	8.2467	5/8	107.6	84.49	31.641	24.850	17.671
11/16	24.56	19.29	7.2227	5.6727	8.4430	11/16	110.0	86.38	32.348	25.406	17.868
3/4	25.71	20.20	7.5625	5.9396	8.6394	3/4	112.4	88.29	33.063	25.967	18.064
13/16	26.90	21.12	7.9102	6.2126	8.8357	13/16	114.9	90.22	33.785	26.535	18.261
7/8	28.10	22.07	8.2656	6.4918	9.0321	7/8	117.4	92.17	34.516	27 109	18.457
15/16	29.34	23.04	8.6289	6.7771	9.2284	15/16	119.9	94.14	35.254	27.688	18.653

TABLE 6.—*Continued.*

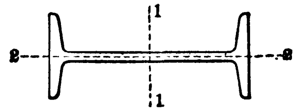
Weights and Areas of Square and Round Bars and Circumferences of Round Bars.

One Cubic Foot of Steel Weighing 489.6 lb.

Thickness or Diameter in Inches.	Weight of ■ Bar One Ft. Long.	Weight of ● Bar One Ft. Long.	Area of ▨ Bar in Sq. Inches.	Area of ◍ Bar in Sq. Inches.	Circumference of ○ Bar in Inches.	Thickness or Diameter in Inches.	Weight of ■ Bar One Ft. Long.	Weight of ● Bar One Ft. Long.	Area of ▨ Bar in Sq. Inches.	Area of ◍ Bar in Sq. Inches.	Circumference of ○ Bar in Inches.
6	122.4	96.14	36.000	28.274	18.850	9	275.4	216.3	81.000	63.617	28.274
1/16	125.0	98.14	36.754	28.866	19.046	1/16	279.3	219.3	82.129	64.505	28.471
1/8	127.6	100.2	37.516	29.465	19.242	1/8	283.2	222.4	83.266	65.397	28.667
3/16	130.2	102.2	38.285	30.069	19.439	3/16	287.0	225.4	84.410	66.296	28.863
1/4	132.8	104.3	39.063	30.680	19.635	1/4	290.9	228.5	85.563	67.201	29.060
5/16	135.5	106.4	39.848	31.296	19.831	5/16	294.9	231.5	86.723	68.112	29.256
3/8	138.2	108.5	40.641	31.919	20.028	3/8	298.9	234.7	87.891	69.029	29.452
7/16	140.9	110.7	41.441	32.548	20.224	7/16	302.8	237.9	89.066	69.953	29.649
1/2	143.6	112.8	42.250	33.183	20.420	1/2	306.8	241.0	90.250	70.882	29.845
9/16	146.5	114.9	43.066	33.824	20.617	9/16	310.9	244.2	91.441	71.818	30.041
5/8	149.2	117.2	43.891	34.472	20.813	5/8	315.0	247.4	92.641	72.760	30.238
11/16	152.1	119.4	44.723	35.125	21.009	11/16	319.1	250.6	93.848	73.708	30.434
3/4	154.9	121.7	45.563	35.785	21.206	3/4	323.2	253.9	95.063	74.662	30.631
13/16	157.8	123.9	46.410	36.450	21.402	13/16	327.4	257.1	96.285	75.622	30.827
7/8	160.8	126.2	47.266	37.122	21.598	7/8	331.6	260.4	97.516	76.589	31.023
15/16	163.6	128.5	48.129	37.800	21.795	15/16	335.8	263.7	98.754	77.561	31.022
7	166.6	130.9	49.000	38.485	21.991	10	340.0	267.0	100.00	78.540	31.416
1/16	169.6	133.2	49.879	39.175	22.187	1/16	344.3	270.4	101.25	79.525	31.612
1/8	172.6	135.6	50.766	39.871	22.384	1/8	348 5	273.8	102.52	80.516	31.809
3/16	175.6	137.9	51.660	40.574	22.580	3/16	352.9	277.1	103.79	81.513	32.005
1/4	178.7	140.4	52.563	41.282	22.777	1/4	357.2	280.6	105.06	82.516	32.201
5/16	181.8	142.8	53.473	41 997	22.973	5/16	361.6	284.0	106.35	83.525	32.398
3/8	184.9	145.3	54.391	42.718	23.169	3/8	366.0	287.4	107.64	84.541	32.594
7/16	188.1	147.7	55.316	43.445	23.366	7/16	370.4	290.9	108.94	85.562	32.790
1/2	191.3	150.2	56.250	44.179	23 562	1/2	374.9	294.4	110.25	86.590	32.987
9/16	194.4	152.7	57.191	44.918	23.758	9/16	379.4	297.9	111.57	87.624	33.183
5/8	197.7	155.2	58.141	45.664	23.955	5/8	383.8	301.4	112.89	88.664	33.379
11/16	200.9	157.8	59.098	46.415	24.151	11/16	388.3	305.0	114.22	89.710	33.576
3/4	204.2	160.3	60 063	47.173	24.347	3/4	392.9	308.6	115.56	90.763	33.772
13/16	207.6	163.0	61.035	47.937	24.544	13/16	397.5	312.2	116.91	91.821	33.968
7/8	210.8	165.6	62.016	48.707	24.740	7/8	402.1	315.8	118.27	92.886	34.165
15/16	214.2	168.2	63.004	49.483	24.936	15/16	406.8	319.5	119.63	93.956	34.361
8	217 6	171.0	64.000	50.265	25.133	11	411.4	323.1	121.00	95.033	34.558
1/16	221.0	173 6	65.004	51.054	25.329	1/16	416.1	326.8	122.38	96.116	34.754
1/8	224.5	176.3	66.016	51.849	25.525	1/8	420.9	330.5	123.77	97.205	34.950
3/16	228.0	179.0	67.035	52.649	25.722	3/16	425.5	334.3	125.16	98.301	35 147
1/4	231.4	181.8	68.063	53.456	25.918	1/4	430.3	337.9	126.56	99.402	35.343
5/16	234.9	184.5	69.098	54.269	26.114	5/16	435.1	341.7	127.97	100.51	35.539
3/8	238.5	187.3	70.141	55.088	26.311	3/8	439.9	345.5	129.39	101.62	35.736
7/16	242.0	190.1	71.191	55.914	26.507	7/16	444.8	349.4	130.82	102.74	35.932
1/2	245.6	193.0	72.250	56.745	26.704	1/2	449.6	353.1	132.25	103.87	36.128
9/16	249.3	195.7	73.316	57.583	26.900	9/16	454.5	357 0	133.69	105.00	36.325
5/8	252.9	198.7	74 391	58.426	27.096	5/8	459.5	360.9	135 14	106.14	36.521
11/16	256.6	201.6	75.473	59.276	27.293	11/16	464.4	364.8	136.60	107.28	36.717
3/4	260.3	204.4	76.563	60.132	27.489	3/4	469.4	368.6	138.06	108.43	36.914
13/16	264.1	207.4	77.660	60.994	27.685	13/16	474.4	372.6	139 54	109 59	37.110
7/8	267.9	210.3	78.766	61.862	27.882	7/8	479 5	376.6	141.02	110.75	37 306
15/16	271.6	213.3	79.879	62.737	28.078	15/16	484.5	380.6	142.50	111.92	37.503

TABLE 7.

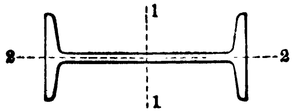
PROPERTIES OF CARNEGIE I BEAMS.



Section Index.	Depth of Beam.	Weight per Foot.	Area of Section.	Width of Flange.	Thickness of Web.	Axis 1–1.			Axis 2–2.		
						I_1	r_1	S_1	I_2	r_2	S_2
	In.	Lb.	In.^{2}	In.	In.	In.^{4}	In.	In.^{3}	In.^{4}	In.	In.^{3}
B 61	27	90.0	26.34	9.000	0.524	2 958.3	10.60	219.1	75.3	1.69	16.7
		120.0	35.13	8.048	0.798	3 010.8	9.26	250.9	84.9	1.56	21.1
B 18	24	115.0	33.67	7.987	0.737	2 940.5	9.35	245.0	82.8	1.57	20.7
		110.0	32.18	7.925	0.675	2 869.1	9.44	239.1	80.6	1.58	20.3
		105.9	30.98	7.875	0.625	2 811.5	9.53	234.3	78.9	1.60	20.0
		100.0	29.25	7.247	0.747	2 371.8	9.05	197.6	48.4	1.29	13.4
		95.0	27.79	7.186	0.686	2 301.5	9.08	191.8	47.0	1.30	13.0
B 1	24	90.0	26.30	7.124	0.624	2 230.1	9.21	185.8	45.5	1.32	12.8
		85.0	24.84	7.063	0.563	2 159.8	9.33	180.0	44.2	1.33	12.5
		79.9	23.33	7.000	0.500	2 087.2	9.46	173.9	42.9	1.36	12.2
B 62	24	74.2	21.70	9.000	0.476	1 950.1	9.48	162.5	61.2	1.68	13.6
B 63	21	60.4	17.68	8.250	0.428	1 235.5	8.36	117.7	43.5	1.57	10.6
		100.0	29.20	7.273	0.873	1 648.3	7.51	164.8	52.4	1.34	14.4
		95.0	27.74	7.200	0.800	1 599.7	7.59	160.0	50.5	1.35	14.0
B 2	20	90.0	26.26	7.126	0.726	1 550.3	7.68	155.0	48.7	1.36	13.7
		85.0	24.80	7.053	0.653	1 501.7	7.78	150.2	47.0	1.38	13.3
		81.4	23.74	7.000	0.600	1 466.3	7.86	146.6	45.8	1.39	13.1
		75.0	21.90	6.391	0.641	1 263.5	7.60	126.3	30.1	1.17	9.4
B 3	20	70.0	20.42	6.317	0.567	1 214.2	7.71	121.4	28.9	1.19	9.2
		65.4	19.08	6.250	0.500	1 169.5	7.83	116.9	27.9	1.21	8.9
		90.0	26.29	7.236	0.796	1 256.5	6.91	139.6	51.9	1.40	14.3
B 19	18	85.0	24.81	7.154	0.714	1 216.6	7.00	135.2	49.8	1.42	14.0
		80.0	23.34	7.072	0.632	1 176.8	7.10	130.8	47.9	1.43	13.6
		75.6	22.04	7.000	0.560	1 141.8	7.20	126.9	46.3	1.45	13.2
		70.0	20.46	6.251	0.711	917.5	6.70	101.9	24.5	1.09	7.8
B 4	18	65.0	18.98	6.169	0.629	877.7	6.80	97.5	23.4	1.11	7.6
		60.0	17.50	6.087	0.547	837.8	6.92	93.1	22.3	1.13	7.3
		54.7	15.94	6.000	0.460	795.5	7.07	88.4	21.2	1.15	7.1
B 64	18	48.2	14.09	7.500	0.380	737.1	7.23	81.9	30.0	1.46	8.0
		75.0	21.85	6.278	0.868	687.2	5.61	91.6	30.6	1.18	9.8
B 6	15	70.0	20.38	6.180	0.770	659.6	5.69	87.9	28.8	1.19	9.3
		65.0	18.91	6.082	0.672	632.1	5.78	84.3	27.2	1.20	8.9
		60.8	17.68	6.000	0.590	609.0	5.87	81.2	26.0	1.21	8.7
		55.0	16.06	5.738	0.648	508.7	5.63	67.8	17.0	1.03	5.9
B 7	15	50.0	14.59	5.640	0.550	481.1	5.74	64.2	16.0	1.05	5.7
		45.0	13.12	5.542	0.452	453.6	5.88	60.5	15.0	1.07	5.4
		42.9	12.49	5.500	0.410	441.8	5.95	58.9	14.6	1.08	5.3
B 65	15	37.3	10.91	6.750	0.332	405.5	6.10	54.1	19.9	1.35	5.9

TABLE 7.—*Continued.*

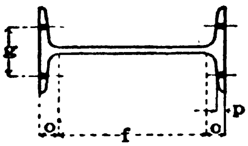
PROPERTIES OF CARNEGIE I BEAMS.

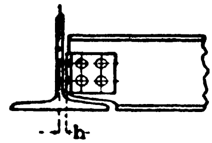


Section Index.	Depth of Beam.	Weight per Foot.	Area of Section.	Width of Flange.	Thickness of Web.	Axis 1-1.			Axis 2-2.		
						I_1	r_1	S_1	I_2	r_2	S_2
	In.	Lb.	$In.^2$	In.	In.	$In.^4$	In.	$In.^3$	$In.^4$	In.	$In.^3$
B 8	12	55.0	16.04	5.600	0.810	319.3	4.46	53.2	17.3	1.04	6.2
		50.0	14.57	5.477	0.687	301.6	4.55	50.3	16.0	1.05	5.8
		45.0	13.10	5.355	0.565	284.1	4.66	47.3	14.8	1.06	5.5
		40.8	11.84	5.250	0.460	268.9	4.77	44.8	13 8	1.08	5.3
B 9	12	35.0	10.20	5.078	0.428	227.0	4.72	37.8	10.0	0.99	3.9
		31.8	9.26	5.000	0.350	215.8	4.83	36.0	9.5	1.01	3.8
B 66	12	27.9	8.15	6.000	0.284	199.4	4.95	33.2	12.6	1.24	4.2
B 10	10	40.0	11.69	5.091	0.741	158.0	3.68	31.6	9.4	0.90	3.7
		35.0	10.22	4.944	0.594	145.8	3.78	29.2	8.5	0.91	3.4
		30.0	8.75	4.797	0.447	133.5	3.91	26.7	7.6	0.93	3.2
		25.4	7.38	4.660	0.310	122.1	4.07	24.4	6.9	0.97	3.0
B 67	10	22.4	6.54	5.500	0.252	113.6	4.17	22.7	9.0	1.17	3.3
B 11	9	35.0	10.22	4.764	0.724	111.3	3.30	24.7	7.3	0.84	3.0
		30.0	8.76	4.601	0.561	101.4	3.40	22.5	6.4	0.85	2.8
		25.0	7.28	4.437	0.397	91.4	3.54	20.3	5.6	0.88	2.5
		21.8	6.32	4.330	0.290	84.9	3.67	18.9	5.2	0.90	2.4
B 12	8	25.5	7.43	4.262	0.532	68.1	3.03	17.0	4.7	0.80	2.2
		23.0	6.71	4.171	0.441	64.2	3.09	16.0	4.4	0.81	2.1
		20.5	5.97	4.079	0.349	60.2	3.18	15.1	4.0	0.82	2.0
		18.4	5.34	4.000	0.270	56.9	3.26	14.2	3.8	0.84	1.9
B 68	8	17.5	5.13	5.000	0.220	58.4	3.38	14.6	6.2	1.10	2.5
B 13	7	20.0	5.83	3.860	0.450	41.9	2.68	12.0	3.1	0.74	1.6
		17.5	5.09	3.755	0.345	38.9	2.77	11.1	2.9	0.76	1.6
		15.3	4.43	3.660	0.250	36.2	2.86	10.4	2.7	0.78	1.5
B 14	6	17.25	5.02	3.565	0.465	26.0	2.28	8.7	2.3	0.68	1.3
		14.75	4.29	3.443	0.343	23.8	2.36	7.9	2.1	0.69	1.2
		12.5	3.61	3.330	0.230	21.8	2.46	7.3	1.8	0.72	1.1
B 15	5	14.75	4.29	3.284	0.494	15.0	1.87	6.0	1.7	0.63	1.0
		12.25	3.56	3.137	0.347	13.5	1.95	5.4	1.4	0.63	0.91
		10.0	2.87	3.000	0.210	12.1	2.05	4.8	1.2	0.65	0.82
B 16	4	10.5	3.05	2.870	0.400	7.1	1.52	3.5	1.0	0.57	0.70
		9.5	2.76	2.796	0.326	6.7	1.56	3.3	0.91	0.58	0.65
		8.5	2.46	2.723	0.253	6.3	1.60	3.2	0.83	0.58	0.61
		7.7	2.21	2.660	0.190	6.0	1.64	3.0	0.77	0.59	0.58
B 17	3	7.5	2.17	2.509	0.349	2.9	1.15	1.9	0.59	0.52	0.47
		6.5	1.88	2.411	0.251	2.7	1.19	1.8	0.51	0.52	0.43
		5.7	1.64	2.330	0.170	2.5	1.23	1.7	0.46	0.53	0.40

TABLE 8.

Elements of Carnegie I Beams.



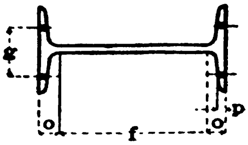


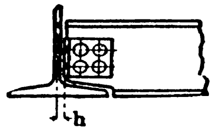
Nominal dimensions are:—flange width and "o" in eighths, web thickness in sixteenths. Gages for connection angles are determined by 1/2 web thickness. Standard gages may be varied if conditions require.

Depth of Beam	Weight per Foot	Flange Width	Web Thickness	1/2 Web Thickness	Gage g	Grip p	Distance f	Distance o	Distance h	Max. Rivet in Flange
In.	Lbs.	In.	In.	In.	In.	In.	In.	In.	In.	In.
27	90.0	9	1/2	1/4	5	3/4	22 1/2	2 1/4	5/16	7/8
	120.0	8	13/16	3/8	5	1 1/8	20 1/4	1 7/8	1/2	
24	115.0	8	3/4	3/8	5	1 1/8	20 1/4	1 7/8	7/16	7/8
	110.0	7 7/8	11/16	5/16	5	1 1/8	20 1/4	1 7/8	7/16	
	105.9	7 7/8	5/8	5/16	5	1 1/8	20 1/4	1 7/8	3/8	
	100.0	7 1/4	3/4	3/8	4	7/8	20 3/4	1 5/8	7/16	
	95.0	7 1/8	11/16	5/16	4	7/8	20 3/4	1 5/8	7/16	
24	90.0	7 1/8	5/8	5/16	4	7/8	20 3/4	1 5/8	3/8	7/8
	85.0	7 1/8	9/16	5/16	4	7/8	20 3/4	1 5/8	3/8	
	79.9	7	1/2	1/4	4	7/8	20 3/4	1 5/8	5/16	
24	74.2	9	1/2	1/4	4	5/8	20	2	5/16	7/8
21	60.4	8 1/4	7/16	3/16	4	9/16	17 1/2	1 3/4	1/4	7/8
	100.0	7 1/4	7/8	7/16	4	1	16 1/2	1 3/4	1/2	
	95.0	7 1/4	13/16	3/8	4	1	16 1/2	1 3/4	1/2	
20	90.0	7 1/8	3/4	3/8	4	1	16 1/2	1 3/4	7/16	7/8
	85.0	7	5/8	5/16	4	1	16 1/2	1 3/4	3/8	
	81.4	7	5/8	5/16	4	1	16 1/2	1 3/4	3/8	
	75.0	6 3/8	5/8	5/16	4	3/4	17	1 1/2	3/8	
20	70.0	6 3/8	9/16	5/16	4	3/4	17	1 1/2	3/8	7/8
	65.4	6 1/4	1/2	1/4	4	3/4	17	1 1/2	5/16	
	90.0	7 1/4	13/16	3/8	4	1	14 1/2	1 3/4	1/2	
18	85 0	7 1/8	11/16	3/8	4	1	14 1/2	1 3/4	7/16	7/8
	80.0	7 1/8	5/8	5/16	4	1	14 1/2	1 3/4	3/8	
	75.6	7	9/16	1/4	4	1	14 1/2	1 3/4	3/8	
	70.0	6 1/4	11/16	3/8	3 3/4	3/4	15 1/4	1 3/8	7/16	
18	65.0	6 1/8	5/8	5/16	3 3/4	3/4	15 1/4	1 3/8	3/8	7/8
	60 0	6 1/8	9/16	1/4	3 3/4	3/4	15 1/4	1 3/8	3/8	
	54.7	6	7/16	1/4	3 3/4	3/4	15 1/4	1 3/8	5/16	
18	48.2	7 1/2	3/8	3/16	3 3/4	1/2	14 3/4	1 5/8	1/4	7/8
	75.0	6 1/4	7/8	7/16	3 1/2	7/8	11 3/4	1 5/8	1/2	
15	70.0	6 1/8	3/4	3/8	3 1/2	7/8	11 3/4	1 5/8	7/16	3/4
	65.0	6 1/8	11/16	5/16	3 1/2	7/8	11 3/4	1 5/8	7/16	
	60.8	6	9/16	5/16	3 1/2	7/8	11 3/4	1 5/8	3/8	
	55.0	5 3/4	5/8	5/16	3 1/2	5/8	12 1/2	1 1/4	3/8	
15	50.0	5 5/8	9/16	1/4	3 1/2	5/8	12 1/2	1 1/4	3/8	3/4
	45.0	5 1/2	7/16	1/4	3 1/2	5/8	12 1/2	1 1/4	5/16	
	42.9	5 1/2	7/16	3/16	3 1/2	5/8	12 1/2	1 1/4	1/4	
15	37.3	6 3/4	5/16	3/16	3 1/2	7/16	12 1/4	1 3/8	1/4	3/4

TABLE 8.—*Continued.*

ELEMENTS OF CARNEGIE I BEAMS.





Nominal dimensions are:—flange width and "o" in eighths, web thickness in sixteenths. Gages for connection angles are determined by 1/2 web thickness. Standard gages may be varied if conditions require.

Depth of Beam	Weight per Foot	Flange Width	Web Thick-ness	1/2 Web Thick-ness	Gage g	Grip p	Distance f	Distance o	Distance h	Max. Rivet in Flange
In.	Lbs.	In.	In.	In.	In.	In.	In.	In.	In.	In.
12	55.0	5 5/8	13/16	3/8	3 1/2	3/4	9 1/4	1 3/8	1/2	3/4
	50.0	5 1/2	11/16	5/16	3 1/2	3/4	9 1/4	1 3/8	7/16	
	45.0	5 3/8	9/16	5/16	3	3/4	9 1/4	1 3/8	3/8	
	40.8	5 1/4	7/16	1/4	3	3/4	9 1/4	1 3/8	5/16	
12	35.0	5 1/8	7/16	3/16	3	9/16	9 3/4	1 1/8	5/16	3/4
	31.8	5	3/8	3/16	3	9/16	9 3/4	1 1/8	1/4	
12	27.9	6	5/16	1/8	3	7/16	9 1/2	1 1/4	3/16	3/4
10	40.0	5 1/8	3/4	3/8	2 3/4	1/2	8	1	7/16	3/4
	35.0	5	5/8	5/16	2 3/4	1/2	8	1	3/8	
	30.0	4 3/4	7/16	1/4	2 3/4	1/2	8	1	5/16	
	25.4	4 5/8	5/16	1/8	2 3/4	1/2	8	1	1/4	
10	22.4	5 1/2	1/4	1/8	2 3/4	3/8	7 3/4	1 1/8	3/16	3/4
9	35.0	4 3/4	3/4	3/8	2 1/2	1/2	7	1	7/16	3/4
	30.0	4 5/8	9/16	1/4	2 1/2	1/2	7	1	3/8	
	25.0	4 1/2	3/8	3/16	2 1/2	1/2	7	1	1/4	
	21.8	4 3/8	5/16	1/8	2 1/2	1/2	7	1	3/16	
8	25.5	4 1/4	9/16	1/4	2 1/4	1/2	6 1/4	7/8	5/16	3/4
	23.0	4 1/8	7/16	1/4	2 1/4	7/16	6 1/4	7/8	5/16	
	20.5	4 1/8	3/8	3/16	2 1/4	7/16	6 1/4	7/8	1/4	
	18.4	4	1/4	1/8	2 1/4	7/16	6 1/4	7/8	3/16	
8	17.5	5	1/4	1/8	2 1/4	3/8	6	1	3/16	3/4
7	20.0	3 7/8	7/16	1/4	2 1/4	3/8	5 1/4	7/8	5/16	5/8
	17.5	3 3/4	3/8	3/16	2 1/4	3/8	5 1/4	7/8	1/4	
	15.3	3 5/8	1/4	1/8	2 1/4	3/8	5 1/4	7/8	3/16	
6	17.25	3 5/8	7/16	1/4	2	3/8	4 1/2	3/4	5/16	5/8
	14.75	3 1/2	3/8	3/16	2	3/8	4 1/2	3/4	1/4	
	12.5	3 3/8	1/4	1/8	2	3/8	4 1/2	3/4	3/16	
5	14.75	3 3/8	1/2	1/4	1 3/4	3/8	3 1/2	3/4	5/16	1/2
	12.25	3 1/8	3/8	3/16	1 3/4	3/8	3 1/2	3/4	1/4	
	10.0	3	3/16	1/8	1 3/4	3/8	3 1/2	3/4	3/16	
4	10.5	2 7/8	5/8	3/16	1 1/2	5/16	2 3/4	5/8	1/4	1/2
	9.5	2 3/4	5/16	3/16	1 1/2	5/16	2 3/4	5/8	1/4	
	8.5	2 3/4	1/4	1/8	1 1/2	5/16	2 3/4	5/8	3/16	
	7.7	2 5/8	3/16	1/8	1 1/2	5/16	2 3/4	5/8	3/16	
3	7.5	2 1/2	3/8	3/16	1 1/2	5/16	1 3/4	5/8	1/4	3/8
	6.5	2 3/8	1/4	1/8	1 1/2	5/16	1 3/4	5/8	3/16	
	5.7	2 3/8	3/16	1/16	1 1/2	5/16	1 3/4	5/8	1/8	

TABLE 9.

Maximum Moments and Web Resistances.

Carnegie I Beams.

Depth of Beam.	Weight per Foot.	Thickness of Web.	Maximum Bending Moment.	Web Resistance.			Minimum End Bearing.	End Reaction, a = 3½″.
				Web Shear.	Minimum Span.	Web. Buckling.		
d		t	M^{max}	V		f_b	a	R
In.	Lb.	In.	Ft.-Lb.	Lb.	Ft.	Lb. per Sq. In.	In.	Lb.
27	90.0	.524	292 180	141 480	8.25	10 080	20.04	54 000
	120.0	.798	334 530	191 520	6.99	13 790	11.40	104 560
24	115.0	.737	326 730	176 880	7.39	13 360	11.96	93 550
	110.0	.675	318 790	162 000	7.87	12 840	12.69	82 350
	105.9	.625	312 390	150 000	8.33	12 350	13.44	73 320
	100.0	.747	263 530	179 280	5.88	13 430	11.87	95 330
	95.0	.686	255 720	164 640	6.21	12 940	12.55	84 320
24	90.0	.624	247 790	149 760	6.62	12 340	13.45	73 130
	85.0	.563	239 980	135 120	7.10	11 620	14.66	62 120
	79.9	.500	231 920	120 000	7.73	10 680	16.46	50 750
24	74.2	.476	216 680	114 240	7.59	10 270	17.38	46 420
21	60.4	.428	156 890	89 880	6.98	10 510	14.74	49 530
	100.0	.873	219 780	174 600	5.04	15 030	8.31	111 550
	95.0	.800	213 290	160 000	5.33	14 670	8.63	99 750
20	90.0	.726	206 710	145 200	5.69	14 230	9.06	87 810
	85.0	.653	200 220	130 600	6.13	13 700	9.60	76 010
	81.4	.600	195 510	120 000	6.52	13 230	10.12	67 450
	75.0	.641	168 470	128 200	5.26	13 590	9.71	74 070
20	70.0	.567	161 890	113 400	5.71	12 890	10.52	62 130
	65.4	.500	155 930	100 000	6.24	12 070	11.57	51 300
	90.0	.796	186 140	143 280	5.20	15 090	7.43	96 080
18	85.0	.714	180 240	128 520	5.61	14 630	7.80	83 580
	80.0	.632	174 340	113 760	6.13	14 070	8.29	71 140
	75.6	.560	169 150	100 800	6.71	13 440	8.89	60 210
	70.0	.711	135 930	127 980	4.25	14 620	7.81	83 160
18	65.0	.629	130 030	113 220	4.59	14 050	8.31	70 690
	60.0	.547	124 120	98 460	5.04	13 310	9.03	58 230
	54.7	.460	117 860	82 800	5.69	12 230	10.22	45 010
18	48.2	.380	109 200	68 400	6.39	10 810	12.16	32 850
	75.0	.868	122 170	130 200	3.75	16 010	5.62	100 730
15	70.0	.770	117 270	115 500	4.06	15 630	5.85	87 230
	65.0	.672	112 370	100 800	4.46	15 130	6.16	73 730
	60.8	.590	108 270	88 500	4.89	14 600	6.53	62 430
	55.0	.648	90 430	97 200	3.72	14 990	6.26	70 430
15	50.0	.550	85 530	82 500	4.15	14 280	6.76	56 930
	45.0	.452	80 620	67 800	4.76	13 250	7.57	43 430
	42.9	.410	78 530	61 500	5.11	12 670	8.09	37 650
15	37.3	.332	72 090	49 800	5.79	11 170	9.68	26 890

See Table 10.

TABLE 9.—*Continued.*

Maximum Moments and Web Resistances.

Carnegie I Beams.

Depth of Beam.	Weight per Foot.	Thickness of Web.	Maximum Bending Moment.	Web Resistance. Web Shear.	Web Resistance. Minimum Span.	Web Resistance. Web Buckling.	Minimum End Bearing.	End Reaction, a = 3½″.
d		t	M^{max}	V		f_b	a	R
In.	Lb.	In.	Ft.-Lb.	Lb.	Ft.	Lb. per Sq. In.	In.	Lb.
	55.0	.810	70 970	97 200	2.92	16 430	4.30	86 530
12	50.0	.687	67 030	82 440	3.25	15 970	4.51	71 330
	45.0	.565	63 130	67 800	3.72	15 320	4.83	56 270
	40.8	.460	59 770	55 200	4.33	14 480	5.29	43 300
12	35.0	.428	50 460	51 360	3.93	14 150	5.48	39 350
	31.8	.350	47 960	42 000	4.57	13 060	6.19	29 710
12	27.9	.284	44 310	34 080	5.20	11 680	7.27	21 570
	40.0	.741	42 140	74 100	2.27	16 660	3.50	74 080
10	35.0	.594	38 870	59 400	2.62	16 090	3.72	57 330
	30.0	.447	35 600	44 700	3.19	15 120	4.11	40 560
	25.4	.310	32 560	31 000	4.20	13 410	4.96	24 950
10	22.4	.252	30 300	25 200	4.81	12 120	5.75	18 330
	35.0	.724	32 970	65 160	2.02	16 850	3.09	70 130
9	30.0	.561	30 040	50 490	2.38	16 220	3.30	52 320
	25.0	.397	27 090	35 730	3.03	15 070	3.72	34 410
	21.8	.290	25 160	26 100	3.86	13 620	4.36	22 720
	25.5	.532	22 680	42 560	2.13	16 400	2.88	47 970
8	23.0	.441	21 390	35 280	2.43	15 860	3.05	38 460
	20.5	.349	20 080	27 920	2.88	15 030	3.32	28 850
	18.4	.270	18 960	21 600	3.51	13 870	3.77	20 590
8	17.5	.220	19 460	17 600	4.42	12 700	4.30	15 370
	20.0	.450	15 980	31 500	2.03	16 310	2.54	38 520
7	17.5	.345	14 840	24 150	2.46	15 490	2.77	28 050
	15.3	.250	13 800	17 500	3.15	14 150	3.20	18 570
	17.25	.465	11 560	27 900	1.66	16 770	2.08	38 980
6	14.75	.343	10 590	20 580	2.06	15 970	2.26	27 390
	12.50	.230	9 680	13 800	2.81	14 480	2.64	16 650
	14.75	.494	8 030	24 700	1.30	17 250	1.65	40 470
5	12.25	.347	7 210	17 350	1.66	16 510	1.78	27 200
	10.0	.210	6 450	10 500	2.46	14 880	2.11	14 840
	10.5	.400	4 720	16 000	1.18	17 270	1.32	31 080
4	9.5	.326	4 460	13 040	1.37	16 870	1.37	24 750
	8.5	.253	4 200	10 120	1.66	16 260	1.46	18 510
	7.7	.190	3 980	7 600	2.09	15 350	1.61	13 120
	7.5	.349	2 560	10 470	0.98	17 510	0.96	25 970
3	6.5	.251	2 370	7 530	1.26	16 930	1.02	18 060
	5.7	.170	2 210	5 100	1.73	15 950	1.13	11 520

TABLE 10.

Maximum Moments and Web Resistances.

Carnegie I Beams and Channels.

Buckling Values of Beam Webs. A series of experiments have been carried out on beams of various depths and web thicknesses to arrive at a basis for a simpler method of computation to use in the investigation of the safe buckling resistance of beams with unsupported webs, and from these experiments the following formulas have been deduced:

$$\text{Safe end reaction } R = f_b \times t(a + d/4)$$

$$\text{Safe interior load } W = 2f_b \times t(a^1 + d/4)$$

In these formulas R is the end reaction, W the concentrated load, t the web thickness, d the depth of the beam, a^1 half the distance over which the concentrated load is applied and a the whole distance over which the end reaction is applied, while f_b is the safe resistance of the web to buckling in pounds per square inch by the formula
$19{,}000 - 100d/2r(d/2 = l$ in column formula $19{,}000 - 100l/r) = 19{,}000 - 173d/t$.

The first formula is general and applies to any condition of loading. The second formula is for a single load concentrated at the center of a span; it can be extended for a system of concentrated loads provided the sum of the distances a^1 is not less than a.

The tables give for beams and channels with unsupported webs:

1. Allowed web resistance f_b, in pounds per square inch computed from this compression formula.

2. The distance a, or the distance over which the end reaction must be distributed when the shearing stress, V, in the web is the maximum allowable of 10,000 pounds per square inch.

3. The allowable end reaction, R, when a is taken at $3\frac{1}{2}$ in. which is the usual length of beam actually resting on the 4-in. angles ordinarily used in building construction for beam seats.

4. The allowable shear V, on the gross area of beam or channel webs at 10,000 pounds per square inch.

Maximum Bending Moments. In addition to the maximum loads on beams and channels as computed from the web resistance, the tables also give maximum bending moments in foot pounds, based on an allowable fiber stress of 16,000 pounds per square inch. These maximum bending moments may be used on inspection instead of the table of properties to ascertain the proper size section to be used in any particular instance.

TABLE 11.

Percent of Tabular Safe Loads for Beams and Channels Without Lateral Support.

Authority.	Ratio of Span, or Distance Between Lateral Supports, to Flange Width.																		
	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
Cambria	100	100	99	93	87	80	73	67	61	56	51	47	43	39	36	33	30	28	26
Am. B. Co.	100	91	81	72	63	53	44	Ratios above not allowed by American Bridge Co.											

The tabular safe loads should be reduced in accordance with the ratios given in the above table in order to insure that the stresses in the compression flanges should not exceed the allowed unit stress.

TABLE 12.

Safe Loads, in Tons, and Deflections, Carnegie I Beams.

American Bridge Company Standards.

Size.	Weight per Foot, Pounds.	Length of Span in Feet.															
		10	11	12	13	14	15	16	17	18	20	22	24	26	28	30	32
27″	90.	117	106	97	90	83	78	73	68	65	58	53	49	45	42	39	36
	Def.	*.06*	*.08*	*.09*	*.10*	*.12*	*.14*	*.16*	*.18*	*.20*	*.25*	*.30*	*.35*	*.42*	*.48*	*.55*	*.63*
24″	120.	134	121	111	102	96	89	84	79	74	67	62	56	51	48	45	42
	115.	130	119	110	101	94	88	82	77	73	65	59	54	50	47	44	41
	110.	127	116	106	98	91	85	80	75	72	64	58	53	49	46	43	40
	105.9	125	113	104	96	89	83	78	73	69	62	57	52	48	45	41	39
	100.	105	96	88	81	76	71	66	62	59	53	48	44	41	38	35	33
	95.	102	93	86	79	73	68	64	60	57	51	47	43	39	37	34	32
	90.	99	90	83	77	71	66	62	59	55	50	45	41	38	36	33	31
	85.	96	87	80	74	69	64	60	57	54	48	44	40	37	34	32	30
	79.9	93	84	77	71	66	62	58	55	52	46	42	39	36	33	31	29
	74.4	86	79	72	66	62	58	54	51	48	43	39	36	33	31	29	27
	Def.	*.07*	*.08*	*.10*	*.12*	*.14*	*.16*	*.18*	*.20*	*.22*	*.28*	*.33*	*.40*	*.47*	*.54*	*.62*	*.71*
21″	60.4	63	57	50	48	45	42	39	37	35	31	28	26	24	22	21	20
	Def.	*.08*	*.10*	*.12*	*.13*	*.15*	*.18*	*.20*	*.23*	*.25*	*.32*	*.38*	*.45*	*.53*	*.62*	*.71*	*.81*
20″	100.	88	80	73	68	63	59	55	52	49	44	40	37	34	32	29	28
	95.	85	78	71	66	61	57	54	50	48	43	39	36	33	31	28	27
	90.	83	75	69	64	59	55	52	49	46	41	38	35	32	30	28	26
	85.	80	73	67	62	57	54	50	47	45	40	37	34	31	29	27	25
	81.4	78	71	65	60	56	52	49	46	43	39	36	33	30	28	26	24
	75.	67	61	56	52	48	45	42	40	38	34	31	28	26	24	23	21
	70.	65	59	54	50	46	43	41	38	36	32	30	27	25	23	22	20
	65.4	62	57	52	48	45	42	39	37	35	31	28	26	24	22	21	19
	Def.	*.08*	*.10*	*.12*	*.14*	*.16*	*.19*	*.21*	*.24*	*.27*	*.33*	*.40*	*.48*	*.56*	*.65*	*.74*	*.85*
18″	90.	74	68	62	57	53	49	46	43	41	37	33	31	28	26	24	23
	85.	72	65	60	55	51	48	45	42	40	36	32	30	27	25	24	22
	80.	70	63	58	53	50	46	43	41	38	35	31	29	26	25	23	21
	75.	67	61	56	52	48	45	42	39	37	33	30	28	26	24	22	21
	70.	54	49	45	42	39	36	34	32	30	27	25	23	21	19	18	17
	65.	52	47	44	40	37	35	33	31	29	26	24	21	20	19	17	16
	60.	50	45	42	38	36	33	31	29	28	25	23	21	19	18	17	16
	54.7	47	43	39	36	34	31	29	28	26	24	21	20	18	17	16	15
	48.2	44	40	36	33	31	29	27	25	24	22	20	18	17	16	15	14
	Def.	*.09*	*.11*	*.13*	*.16*	*.18*	*.21*	*.24*	*.27*	*.30*	*.37*	*.45*	*.53*	*.62*	*.72*	*.83*	*.94*
15″	75.	49	45	41	38	35	33	31	29	27	24	22	20	19	18	16	....
	70.	47	43	39	36	34	31	29	28	26	23	21	20	18	17	16	....
	65.	45	41	38	35	32	30	28	27	25	22	21	19	17	16	15	....
	60.8	43	39	36	33	31	29	27	25	24	21	20	18	17	15	14	....
	55.	36	33	30	28	26	24	23	21	20	18	17	15	14	13	12	....
	50.	34	31	29	26	25	23	21	20	19	17	16	14	13	12	11	....
	45.	32	29	27	25	23	22	20	19	18	16	15	14	12	12	11	....
	42.9	31	29	26	24	22	21	20	18	17	16	14	13	12	11	10	....
	37.3	29	26	24	22	21	19	18	17	16	14	13	12	11	10	9.6	....
	Def.	*.11*	*.13*	*.16*	*.19*	*.22*	*.25*	*.28*	*.32*	*.36*	*.44*	*.53*	*.64*	*.75*	*.87*	*.99*	....

The figures give the safe uniform load in tons, based on extreme fiber stress of 16,000 lb., or the end reactions from safe uniform load in thousands of pounds.

For load concentrated at center, use one-half of figures given for allowable load and four-fifths values given for deflection. Figures for deflections are given in inches.

For figures at right of heavy zigzag lines, deflections are considered excessive for plastered ceilings.

TABLE 13.

Safe Loads, in Tons, and Deflections, Carnegie I Beams.

American Bridge Company Standards.

Size.	Weight per Foot, Pounds.	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	20	22	24
		Length of Span in Feet.																	
12″	55.	....	57	47	40	35	31	28	26	24	22	20	19	18	17	16	14	13	12
	50.	....	53	44	38	33	30	27	25	22	21	19	18	17	16	15	13	12	11
	45.	....	50	42	30	31	28	25	23	21	20	18	17	16	15	14	13	12	11
	40.8	....	48	40	34	30	26	24	22	20	18	17	16	15	14	13	12	11	10
	35.	....	40	34	29	25	22	20	18	17	16	14	14	13	12	11	10	9.2	8.5
	31.8	....	38	32	27	24	21	19	17	16	15	14	13	12	11	11	10	8.7	8.0
	27.9	....	35	30	25	22	20	18	16	15	14	13	12	11	10	9	9	8	7.4
	Def.	....	*.035*	*.05*	*.07*	*.09*	*.11*	*.14*	*.17*	*.20*	*.23*	*.27*	*.31*	*.35*	*40*	*.45*	*.55*	*.67*	*.79*
10″	40.	....	34	28	24	21	19	17	15	14	13	12	11	11	10	9.4	8.5	7.7	7.1
	35.	....	31	26	22	20	17	16	14	13	12	11	10	9.8	9.2	8.7	7.8	7.1	6.5
	30.	....	29	24	20	18	16	14	13	12	11	10	9.5	8.9	8.4	8.0	7.2	6.5	6.0
	25.4	....	26	22	19	16	14	13	12	11	10	9.3	8.7	8.1	7.7	7.2	6 5	5.9	5.4
	22.4	....	24	20	17	15	13	12	11	10	9	8.6	8.1	7.5	7.1	6.4	6.0	5.5	5.0
	Def.	....	*.04*	*.06*	*.08*	*.11*	*.13*	*.17*	*.20*	*.24*	*.28*	*.32*	*.37*	*.42*	*.48*	*.51*	*.66*	*.80*	*.05*
9″	35.	....	26	22	19	17	15	13	12	11	10	9.5	8.8	8.3	7.8	7.4	6.6	6.0	5.5
	30.	....	24	20	17	15	13	12	11	10	9.3	8.6	8.1	7.5	7.1	6.7	6.0	5.5	5.0
	25.	....	22	18	16	14	12	11	9.9	9.1	8.4	7.8	7.3	6.8	6.4	6.1	5.4	5.0	4.5
	21.8	....	20	17	14	13	11	10	9.2	8.4	7.7	7.2	6.7	6.3	5.9	5.6	5.0	4.6	4.2
	Def.	..	*.05*	*.07*	*.09*	*.12*	*.15*	*.18*	*.22*	*.27*	*.31*	*.36*	*.41*	*.47*	*.53*	*.60*	*.74*	*.80*	*1.1*
8″	25.5	....	18	15	13	11	10	9.1	8.3	7.6	7.0	6.5	6.1	5.7	5.4	5.1	4.6	4.2	3.8
	23.	....	17	14	12	11	9.6	8.6	7.8	7.2	6.6	6.1	5.7	5.4	5.1	4.8	4.3	3.9	3.6
	20.5	....	16	13	12	10	9.0	8.1	7.3	6.7	6.2	5.8	5.4	5.1	4.8	4.5	4.0	3.7	3.4
	18.4	....	15	13	11	9.5	8.4	7.6	6.9	6.3	5.8	5.4	5.1	4.7	4.5	4.2	3.8	3.4	3.2
	17.5	....	15	13	11	9.7	8.6	7.8	7.1	6.5	6.0	5.5	5.2	4.9	4.6	4.3	3.9	3.5	3.2
	Def.	....	*.05*	*.07*	*.10*	*.13*	*.17*	*.21*	*.25*	*.30*	*.35*	*.41*	*.47*	*.53*	*.60*	*.67*	*.83*	*1.0*	*1.2*
7″	20.	....	13	11	9.2	8.0	7.1	6.4	5.8	5.4	4.9	4.6	4.3	4.0	3.8	3.6	3.2	2.9	2.7
	17.5	....	12	10	8.5	7.5	6.6	6.0	5.4	5.0	4.6	4.3	4.0	3.7	3.5	3.3	3.0	2.7	2.5
	15.3	...	11	9.2	7.9	6.9	6.1	5.5	5.0	4.6	4.3	3.9	3.7	3.5	3.3	3.1	2.8	2.5	2.3
	Def.	....	*.06*	*.09*	*.12*	*.15*	*.19*	*.24*	*.29*	*.34*	*.40*	*.46*	*.53*	*.61*	*.68*	*.77*	*.05*	*1.1*	*1.4*
6″	17.25	12	9.3	7.8	6.6	5.8	5.2	4.7	4.2	3.9	3.6	3.3	3.1	2.9	2.7	...	...	...	...
	14.75	10	8.5	7.1	6.1	5.3	4.7	4.3	3.9	3.6	3.3	3.0	2.8	2.6	2.5	...	...	...	...
	12.5	9.7	7.8	6 5	5.5	4.8	4.3	3.9	3.5	3.2	3.0	2.8	2.6	2.4	2.3	...	...	...	...
	Def.	*.04*	*.07*	*.10*	*.14*	*.18*	*.22*	*.28*	*.33*	*.40*	*.47*	*.54*	*.62*	*71*	*.80*	..	...	...	...
5″	14.75	8.1	6.5	5.4	4.6	4.0	3.6	3.2	2.9	2.7	2.5	2.3	2.2	2.0	1.9	...	...	...	...
	12.25	7.3	5.8	4.8	4.2	3.6	3.2	2.9	2.6	2.4	2.2	2.1	1.9	1.8	1.7	...	...	...	...
	10.0	6.5	5.2	4.3	3.7	3.2	2.9	2.6	2.3	2.2	2.0	1.8	1.7	1.6	1.5	...	...	...	...
	Def.	*.05*	*.08*	*.12*	*.16*	*.21*	*.27*	*.33*	*.40*	*.48*	*.56*	*.65*	*.74*	*.85*	*.96*	...	...	...	...
4″	10.5	4.8	3.8	3.2	2.7	2.4	2.1	1.9	1.7	1.6	....	....	....	....	...	...	...	...	...
	9.5	4.5	3.6	3.0	2.6	2.3	2.0	1.8	1.6	1.5	....	....	....	....	...	...	...	...	...
	8.5	4.2	3.4	2.8	2.4	2.1	1.9	1.7	1.5	1.4	....	....	....	....	...	...	...	...	...
	7.7	4.0	3.2	2.7	2.3	2.0	1.8	1.6	1.4	1.3	....	....	....	....	...	...	...	...	...
	Def.	*.07*	*.10*	*.15*	*.20*	*.26*	*.33*	*.41*	*.50*	*.60*	....	....	....	....	...	...	...	...	...
3″	7.5	2.6	2.1	1.7	1.5	1.3	1.2	1.0	.94	.86	....	....	....	....	...	...	...	...	...
	6.5	2.4	1.9	1.6	1.4	1.2	1.1	.96	.87	.80	....	....	....	....	...	...	...	...	...
	5.7	2 2	1.8	1.5	1.3	1.1	.98	.88	.80	.73	....	....	....	....	...	...	...	...	...
	Def.	*.09*	*.14*	*.20*	*.27*	*.35*	*.45*	*.55*	*.67*	*.80*	....	....	....	....	...	...	...	...	...

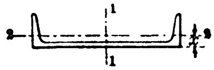
The figures give the safe uniform load in tons, based on extreme fibre stress of 16,000 lb., or the end reactions from safe uniform load in thousands of pounds.

For load concentrated at center, use one-half of figures given for safe loads and four-fifths of the values given for deflections. Figures for deflections are given in inches.

For figures at right of heavy zigzag lines, deflections are excessive for plastered ceilings.

TABLE 14.

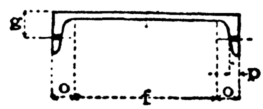
Properties of Carnegie Channels.

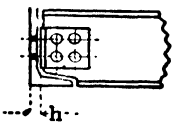


Section Index.	Depth of Channel.	Weight per Foot.	Area of Section.	Width of Flange.	Thickness of Web.	Axis 1-1.			Axis 2-2.			
						I_1	r_1	S_1	I_2	r_2	S_2	x
	In.	Lb.	In.^{2}	In.	In.	In.^{4}	In.	In.^{3}	In.^{4}	In.	In.^{3}	In.
C 1	15	55.0	16.11	3.814	0.814	429.0	5.16	57.2	12.1	0.87	4.1	0.82
		50.0	14.64	3.716	0.716	401.4	5.24	53.6	11.2	0.87	3.8	0.80
		45.0	13.17	3.618	0.618	373.9	5.33	49.8	10.3	0.88	3.6	0.79
		40.0	11.70	3.520	0.520	346.3	5.44	46.2	9.3	0.89	3.4	0.78
		35.0	10.23	3.422	0.422	318.7	5.58	42.5	8.4	0.91	3.2	0.79
		33.9	9.90	3.400	0.400	312.6	5.62	41.7	8.2	0.91	3.2	0.79
C 2	12	40.0	11.73	3.415	0.755	196.5	4.09	32.8	6.6	0.75	2.5	0.72
		35.0	10.26	3.292	0.632	178.8	4.18	29.8	5.9	0.76	2.3	0.69
		30.0	8.79	3.170	0.510	161.2	4.28	26.9	5.2	0.77	2.1	0.68
		25.0	7.32	3.047	0.387	143.5	4.43	23.9	4.5	0.79	1.9	0.68
		20.7	6.03	2.940	0.280	128.1	4.61	21.4	3.9	0.81	1.7	0.70
C 3	10	35.0	10.27	3.180	0.820	115.2	3.34	23.0	4.6	0.67	1.9	0.69
		30.0	8.80	3.033	0.673	103.0	3.42	20.6	4.0	0.67	1.7	0.65
		25.0	7.33	2.886	0.526	90.7	3.52	18.1	3.4	0.68	1.5	0.62
		20.0	5.86	2.739	0.379	78.5	3.66	15.7	2.8	0.70	1.3	0.61
		15.3	4.47	2.600	0.240	66.9	3.87	13.4	2.3	0.72	1.2	0.64
C 4	9	25.0	7.33	2.812	0.612	70.5	3.10	15.7	3.0	0.64	1.4	0.61
		20.0	5.86	2.648	0.448	60.6	3.22	13.5	2.4	0.65	1.2	0.59
		15.0	4.39	2.485	0.285	50.7	3.40	11.3	1.9	0.67	1.0	0.59
		13.4	3.89	2.430	0.230	47.3	3.49	10.5	1.8	0.67	0.97	0.61
C 5	8	21.25	6.23	2.619	0.579	47.6	2.77	11.9	2.2	0.60	1.1	0.59
		18.75	5.49	2.527	0.487	43.7	2.82	10.9	2.0	0.60	1.0	0.57
		16.25	4.76	2.435	0.395	39.8	2.89	9.9	1.8	0.61	0.94	0.56
		13.75	4.02	2.343	0.303	35.8	2.99	9.0	1.5	0.62	0.86	0.56
		11.5	3.36	2.260	0.220	32.3	3.10	8.1	1.3	0.63	0.79	0.58
C 6	7	19.75	5.79	2.509	0.629	33.1	2.39	9.4	1.8	0.56	0.96	0.58
		17.25	5.05	2.404	0.524	30.1	2.44	8.6	1.6	0.56	0.86	0.55
		14.75	4.32	2.299	0.419	27.1	2.51	7.7	1.4	0.57	0.79	0.53
		12.25	3.58	2.194	0.314	24.1	2.59	6.9	1.2	0.58	0.71	0.53
		9.8	2.85	2.090	0.210	21.1	2.72	6.0	0.98	0.59	0.63	0.55
C 7	6	15.5	4.54	2.279	0.559	19.5	2.07	6.5	1.3	0.53	0.73	0.55
		13.0	3.81	2.157	0.437	17.3	2.13	5.8	1.1	0.53	0.65	0.52
		10.5	3.07	2.034	0.314	15.1	2.22	5.0	0.87	0.53	0.57	0.50
		8.2	2.39	1.920	0.200	13.0	2.34	4.3	0.70	0.54	0.50	0.52
C 8	5	11.5	3.36	2.032	0.472	10.4	1.76	4.1	0.82	0.49	0.54	0.51
		9.0	2.63	1.885	0.325	8.8	1.83	3.5	0.64	0.49	0.45	0.48
		6.7	1.95	1.750	0.190	7.4	1.95	3.0	0.48	0.50	0.38	0.49
C 9	4	7.25	2.12	1.720	0.320	4.5	1.47	2.3	0.44	0.46	0.35	0.46
		6.25	1.82	1.647	0.247	4.1	1.50	2.1	0.38	0.45	0.32	0.46
		5.4	1.56	1.580	0.180	3.8	1.56	1.9	0.32	0.45	0.29	0.46
C 10	3	6.0	1.75	1.596	0.356	2.1	1.08	1.4	0.31	0.42	0.27	0.46
		5.0	1.46	1.498	0.258	1.8	1.12	1.2	0.25	0.41	0.24	0.44
		4.1	1.19	1.410	0.170	1.6	1.17	1.1	0.20	0.41	0.21	0.44

TABLE 15.

Elements of Carnegie Channels.





Nominal dimensions are—flange width and "o" in eighths, web thickness in sixteenths. Gages for connection angles are determined by web thickness. Standard gages may be varied if conditions require.

Depth of Channel	Weight per Foot	Flange Width	Web Thickness	½ Web Thickness	Gage g	Grip p	Distance f	Distance o	Distance h	Max. Rivet in Flange
In	Lbs.	In.	In.	In.	In.	In.	In.	In.	In.	In.
15	55.0	3 7/8	13/16	7/16	2 1/2	1 1/16	12 1/4	1 3/8	7/8	7/8
	50.0	3 3/4	11/16	3/8	2 1/2	1 1/16	12 1/4	1 3/8	13/16	
	45.0	3 5/8	5/8	5/16	2	5/8	12 1/4	1 3/8	11/16	
	40.0	3 1/2	1/2	1/4	2	5/8	12 1/4	1 3/8	9/16	
	35.0	3 7/16	7/16	3/16	2	5/8	12 1/4	1 3/8	1/2	
	33.9	3 3/8	3/8	3/16	2	5/8	12 1/4	1 3/8	1/2	
13	50.0	4 3/8	13/16	3/8	3	9/16	10 1/2	1 1/4	7/8	7/8
	45.0	4 1/4	11/16	5/16	2 3/4	9/16	10 1/2	1 1/4	3/4	
	40.0	4 1/8	9/16	1/4	2 3/4	9/16	10 1/2	1 1/4	5/8	
	37.0	4 1/8	1/2	1/4	2 1/2	9/16	10 1/2	1 1/4	9/16	
	35.0	4 1/8	7/16	1/4	2 1/2	9/16	10 1/2	1 1/4	1/2	
	31.8	4	3/8	3/16	2 1/2	9/16	10 1/2	1 1/4	7/16	
12	40.0	3 3/8	3/4	3/8	2	5/8	10	1	13/16	7/8
	35.0	3 1/4	5/8	5/16	2	5/8	10	1	11/16	
	30.0	3 1/8	1/2	1/4	1 3/4	1/2	10	1	9/16	
	25.0	3	3/8	3/16	1 3/4	1/2	10	1	7/16	
	20.7	3	1/4	1/8	1 3/4	1/2	10	1	3/8	
10	35.0	3 1/8	13/16	7/16	1 3/4	1/2	8 1/4	7/8	7/8	3/4
	30 0	3	11/16	5/16	1 3/4	1/2	8 1/4	7/8	3/4	
	25.0	2 7/8	1/2	1/4	1 3/4	1/2	8 1/4	7/8	9/16	
	20.0	2 3/4	3/8	3/16	1 1/2	7/16	8 1/4	7/8	7/16	
	15.3	2 5/8	1/4	1/8	1 1/2	7/16	8 1/4	7/8	5/16	
9	25.0	2 7/8	5/8	5/16	1 1/2	1/2	7 1/4	7/8	11/16	3/4
	20.0	2 5/8	7/16	1/4	1 1/2	1/2	7 1/4	7/8	1/2	
	15.0	2 1/2	5/16	1/8	1 3/8	7/16	7 1/4	7/8	3/8	
	13.4	2 3/8	1/4	1/8	1 3/8	7/16	7 1/4	7/8	5/16	
8	21.25	2 5/8	9/16	5/16	1 1/2	7/16	6 1/4	7/8	11/16	3/4
	18.75	2 1/2	1/2	1/4	1 1/2	7/16	6 1/4	7/8	9/16	
	16.25	2 3/8	3/8	3/16	1 1/2	7/16	6 1/4	7/8	1/2	
	13.75	2 3/8	5/16	1/8	1 3/8	3/8	6 1/4	7/8	3/8	
	11.5	2 1/4	1/4	1/8	1 3/8	3/8	6 1/4	7/8	5/16	
7	19.75	2 1/2	5/8	5/16	1 1/2	7/16	5 1/2	3/4	11/16	5/8
	17.25	2 3/8	1/2	1/4	1 1/2	7/16	5 1/2	3/4	9/16	
	14.75	2 1/4	7/16	3/16	1 1/4	7/16	5 1/2	3/4	1/2	
	12.25	2 1/4	5/16	3/16	1 1/4	3/8	5 1/2	3/4	3/8	
	9.8	2 1/8	3/16	1/8	1 1/4	3/8	5 1/2	3/4	5/16	
6	15.5	2 1/4	9/16	1/4	1 3/8	3/8	4 1/2	3/4	5/8	5/8
	13.0	2 1/8	7/16	1/4	1 3/8	3/8	4 1/2	3/4	1/2	
	10.5	2	5/16	3/16	1 1/8	3/8	4 1/2	3/4	3/8	
	8.2	1 7/8	3/16	1/8	1 1/8	5/16	4 1/2	3/4	1/4	
5	11.5	2	1/2	1/4	1 1/8	5/16	3 3/4	5/8	9/16	1/2
	9.0	1 7/8	5/16	3/16	1 1/8	5/16	3 3/4	5/8	3/8	
	6.7	1 3/4	3/16	1/8	1 1/8	5/16	3 3/4	5/8	1/4	
4	7.25	1 3/4	5/16	3/16	1	5/16	2 3/4	5/8	3/8	1/2
	6.25	1 5/8	1/4	1/8	1	5/16	2 3/4	5/8	5/16	
	5.4	1 5/8	3/16	1/16	1	5/16	2 3/4	5/8	1/4	
3	6.0	1 5/8	3/8	3/16	7/8	1/4	1 3/4	5/8	7/16	1/2
	5.0	1 1/2	1/4	1/8	7/8	1/4	1 3/4	5/8	5/16	
	4.1	1 3/8	3/16	1/16	7/8	1/4	1 3/4	5/8	1/4	

TABLE 16.

Maximum Moments and Web Resistances.

Carnegie Channels.

Depth of Channel.	Weight per Foot.	Thickness of Web.	Maximum Bending Moment.	Web Resistance. Web Shear.	Web Resistance. Minimum Span.	Web Resistance. Web Buckling.	Minimum End Bearing	End Reaction, a = 3½″.
d		t	M_{max}	V		f_b	a	R
In.	Lb.	In.	Ft.-Lb.	Lb.	Ft.	Lb. per Sq. In.	In.	Lb.
	55.0	.814	76 270	122 100	2.50	15 810	5.74	93 290
	50.0	.716	71 420	107 400	2.66	15 370	6.01	79 790
15	45.0	.618	66 470	92 700	2.87	14 800	6.39	66 290
	40.0	.520	61 570	78 000	3.16	14 000	6.96	52 790
	35.0	.422	56 670	63 300	3.58	12 840	7.93	39 290
	33.9	.400	55 570	60 000	3.70	12 510	8.24	36 270
	50.0	.787	64 190	102 310	2.51	16 140	4.81	85 730
	45.0	.673	59 910	87 490	2.74	15 660	5.05	71 120
13	40.0	.560	55 660	72 800	3.06	14 980	5.43	56 620
	37.0	.492	53 110	63 960	3.32	14 420	5.76	47 900
	35.0	.447	51 420	58 110	3.54	13 960	6.06	42 120
	31.8	.375	48 720	48 750	4.00	13 000	6.75	32 900
	40.0	.755	43 670	90 600	1.93	16 250	4.39	79 740
	35.0	.632	39 730	75 840	2.10	15 710	4.64	64 540
12	30.0	.510	35 830	61 200	2.34	14 920	5.04	49 470
	25.0	.387	31 890	46 440	2.75	13 630	5.81	34 280
	20.7	.280	28 470	33 600	3.39	11 570	7.37	21 060
	35.0	.820	30 720	82 000	1.50	16 890	3.42	83 090
	30.0	.673	27 460	67 300	1.63	16 430	3.59	66 330
10	25.0	.526	24 190	52 600	1.84	15 710	3.87	49 570
	20.0	.379	20 920	37 900	2.21	14 430	4.43	32 810
	15.3	.240	17 830	24 000	2.94	11 790	5.98	16 970
	25.0	.612	20 900	55 080	1.52	16 450	3.22	57 900
9	20.0	.448	17 950	40 320	1.78	15 520	3.55	39 980
	15.0	.285	15 010	25 650	2.34	13 530	4.40	22 180
	13.4	.230	14 020	20 700	2.71	12 220	5.11	16 160
	21.25	.579	15 870	46 320	1.37	16 610	2.82	52 880
	18.75	.487	14 570	38 960	1.50	16 160	2.95	43 270
8	16.25	.395	13 260	31 600	1.68	15 490	3.16	33 650
	13.75	.303	11 950	24 240	1.97	14 430	3.55	24 040
	11.5	.220	10 770	17 600	2.45	12 700	4.30	15 370
	19.75	.629	12 590	44 030	1.14	17 070	2.35	56 380
	17.25	.524	11 450	36 680	1.25	16 690	2.44	45 910
7	14.75	.419	10 310	29 330	1.21	16 110	2.60	35 430
	12.25	.314	9 170	21 980	1.67	15 140	2.87	24 950
	9.8	.210	8 030	14 700	2.19	13 220	3.54	14 580
	15.5	.559	8 650	33 540	1.03	17 140	2.00	47 910
6	13.0	.437	7 670	26 220	1.17	16 620	2.11	36 320
	10.5	.314	6 690	18 840	1.42	15 690	2.32	24 630
	8.2	.200	5 780	12 000	1.67	13 800	2.85	13 800
	11.5	.472	5 520	23 600	0.94	17 170	1.66	38 490
5	9.0	.325	4 710	16 250	1.16	16 340	1.81	25 220
	6.7	.190	3 950	9 500	1.67	14 440	2.21	13 030
	7.25	.320	3 030	12 800	0.95	16 840	1.38	24 240
4	6.25	.247	2 770	9 880	1.12	16 200	1.47	18 000
	5.4	.180	2 530	7 200	1.40	15 150	1.64	12 270
	6.0	.356	1 830	10 680	0.68	17 540	0.96	26 540
3	5.0	.258	1 630	7 740	0.84	16 990	1.02	18 630
	4.1	.170	1 450	5 100	1.14	15 950	1.13	11 520

See Table 10.

TABLE 17

Safe Loads, in Tons, and Deflections, Carnegie Channels

American Bridge Company Standards

Size	Weight per Foot, Pounds	Length of Span in Feet 8	9	10	11	12	13	14	15	16	18	20	22	24	26	28	30
15″	55.	38	34	31	28	25	24	22	20	19	17	15	14	13	12	11	10
	50.	36	32	29	26	24	22	20	19	18	16	14	13	12	11	10	9.5
	45.	33	30	27	24	22	21	19	18	17	15	13	12	11	10	9.5	8.9
	40.	31	27	25	22	21	19	18	16	15	14	12	11	10	9.5	8.8	8.2
	35.	28	25	23	21	19	18	16	15	14	13	11	10	9.5	8.8	8.1	7.6
	33.9	28	25	22	20	19	17	16	15	14	12	11	10	9.3	8.6	7.9	7.4
	Def.	*.07*	*.09*	*.11*	*.13*	*.16*	*.19*	*.22*	*.25*	*.28*	*.36*	*.44*	*.53*	*.64*	*.75*	*.87*	*.99*
12″	40.	22	19	18	16	15	13	13	12	11	9.7	8.8	8.0	7.3	6.7	6.3	5.8
	35.	20	18	16	14	13	12	11	10	10	8.9	8.0	7.2	6.6	6.1	5.7	5.3
	30.	18	16	14	13	12	11	10	9.6	9.0	8.0	7.2	6.5	6.0	5.5	5.1	4.8
	25.	16	14	13	12	11	9.9	9.1	8.5	8.0	7.1	6.4	5.8	5.3	4.9	4.6	4.3
	20.7	14	13	11	10	9.5	8.8	8.1	7.6	7.1	6.3	5.7	5.2	4.7	4.4	4.1	3.8
	Def.	*.09*	*.11*	*.14*	*.17*	*.20*	*.23*	*.27*	*.31*	*.35*	*.45*	*.55*	*.67*	*.79*	*.93*	*1.1*	*1.2*
10″	35.	15	14	12	11	10	9.5	8.8	8.2	7.7	6.8	6.2	5.6	5.1	4.7	4.4	4.1
	30.	14	12	11	10	9.2	8.5	7.9	7.3	6.9	6.1	5.5	5.0	4.6	4.2	3.9	3.7
	25.	12	11	9.7	8.8	8.1	7.5	6.9	6.5	6.1	5.4	4.9	4.4	4.0	3.7	3.5	3.2
	20.	11	9.3	8.4	7.6	7.0	6.5	6.0	5.6	5.3	4.7	4.2	3.8	3.5	3.2	3.0	2.8
	15.3	8.9	7.9	7.1	6.5	5.9	5.5	5.1	4.8	4.5	4.0	3.6	3.2	3.0	2.7	2.6	2.4
	Def.	*.11*	*.13*	*.17*	*.20*	*.24*	*.28*	*.32*	*.37*	*.42*	*.54*	*.66*	*.80*	*.95*	*1.1*	*1.3*	*1.5*
9″	25.	10	9.3	8.4	7.6	7.0	6.4	6.0	5.6	5.2	4.7	4.2	3.8	3.5	3.2	3.0	2.8
	20	9.0	8.0	7.2	6.6	6.0	5.5	5.1	4.8	4.5	4.0	3.6	3.3	3.0	2.8	2.6	2.4
	15.	7.5	6.7	6.0	5.5	5.0	4.6	4.3	4.0	3.8	3.3	3.0	2.7	2.5	2.3	2.2	2.0
	13.4	7.0	6.2	5.6	5.1	4.7	4.3	4.0	3.7	3.5	3.1	2.8	2.6	2.3	2.2	2.0	1.9
	Def.	*.12*	*.15*	*.18*	*.22*	*.27*	*.31*	*.36*	*.41*	*.47*	*.60*	*.74*	*.89*	*1.1*	*1.2*	*1.4*	*1.7*

The figures give the safe uniform load in tons, based on extreme fiber stress of 16,000 lb., or the end reactions from safe uniform load in thousands of pounds.

For load concentrated at center, use one-half of figures given for safe loads and four-fifths of the values given for deflections.

Figures for deflections are given in inches.

For figures at right of heavy zigzag lines, deflections are considered excessive for plastered ceilings.

TABLE 17.—*Continued*

SAFE LOADS, IN TONS, AND DEFLECTIONS, CARNEGIE CHANNELS
AMERICAN BRIDGE COMPANY STANDARDS

Size	Weight per Foot, Pounds	Length of Span in Feet 5	6	7	8	9	10	11	12	13	14	15	16	18	20	22	24
8″	21.25	13	11	9.1	7.9	7.1	6.4	5.8	5.3	4.9	4.6	4.2	4.0	....	....	....	....
	18.75	12	9.7	8.4	7.3	6.5	5.8	5.3	4.9	4.5	4.2	3.9	3.7	....	....	....	....
	16.25	11	8.9	7.6	6.7	5.9	5.3	4.8	4.4	4.1	3.8	3.5	3.3	....	....	....	....
	13.75	9.6	8.0	6.9	6.0	5.3	4.8	4.4	4.0	3.7	3.4	3.2	3.0	....	....	....	....
	11.5	8.6	7.2	6.2	5.4	4.8	4.3	3.9	3.6	3.3	3.1	2.9	2.7	....	....	....	....
	Def.	*.05*	*.07*	*.10*	*.13*	*.17*	*.21*	*.25*	*.30*	*.35*	*.41*	*.47*	*.53*	....	....	....	....
7″	19.75	10	8.4	7.2	6.3	5.6	5.1	4.6	4.2	3.9	3.6	3.4	3.2	....	....	....	....
	17.25	9.2	7.7	6.6	5.8	5.1	4.6	4.2	3.8	3.5	3.3	3.1	2.9	....	....	....	....
	14.75	8.3	6.9	5.9	5.2	4.6	4.1	3.8	3.5	3.2	3.0	2.8	2.6	....	....	....	....
	12.25	7.4	6.1	5.3	4.6	4.1	3.7	3.4	3.1	2.8	2.6	2.5	2.3	....	....	....	....
	9.8	6.7	5.6	4.8	4.2	3.7	3.3	3.0	2.8	2.6	2.4	2.2	2.1	....	....	....	....
	Def.	*.06*	*.09*	*.12*	*.15*	*.19*	*.24*	*.29*	*.34*	*.40*	*.46*	*.53*	*.61*	....	....	....	....
6″	15.5	7.0	5.8	5.0	4.3	3.9	3.5	3.2	2.9	2.7	2.5	2.3	2.2	....	....	....	....
	13.	6.2	5.1	4.4	3.9	3.4	3.1	2.8	2.6	2.4	2.2	2.1	1.9	....	....	....	....
	10.5	5.4	4.5	3.8	3.4	3.0	2.7	2.4	2.2	2.1	1.9	1.8	1.7	....	....	....	....
	8.2	4.6	3.9	3.3	2.9	2.6	2.3	2.1	1.9	1.8	1.7	1.5	1.4	....	....	....	....
	Def.	*.07*	*.10*	*.14*	*.18*	*.22*	*.28*	*.33*	*.40*	*.47*	*.54*	*.62*	*.71*	....	....	....	....
5″	11.5	4.4	3.7	3.2	2.8	2.5	2.2	2.0	1.9	1.7	1.6	1.5	1.4	....	....	....	....
	9.	3.8	3.2	2.7	2.4	2.1	1.9	1.7	1.6	1.5	1.4	1.3	1.2	....	....	....	....
	6.7	3.2	2.6	2.3	2.0	1.8	1.6	1.4	1.3	1.2	1.1	1.0	.99	....	....	....	....
	Def.	*.08*	*.12*	*.16*	*.21*	*.27*	*.33*	*.40*	*.48*	*.56*	*.65*	*.74*	*.85*	....	....	....	....
4″	7.25	2.4	2.0	1.7	1.5	1.4	1.2	1.1	1.0	.94	.87	.81	.76	....	....	....	....
	6.25	2.2	1.9	1.6	1.4	1.2	1.1	1.0	.93	.86	.80	.74	.70	....	....	....	....
	5.4	2.0	1.7	1.4	1.3	1.1	1.0	.92	.84	.78	.72	.67	.63	....	....	....	....
	Def.	*.10*	*.15*	*.20*	*.26*	*.34*	*.41*	*.50*	*.60*	*.70*	*.81*	*.93*	*1.1*	....	....	....	....
3″	6.	1.5	1.2	1.1	.92	.82	.74	.67	.61	.57	.53	.49	.46	....	....	....	....
	5.	1.3	1.1	.94	.82	.73	.66	.60	.55	.50	.47	.44	.41	....	....	....	....
	4.1	1.2	.97	.83	.73	.64	.58	.53	.48	.45	.41	.39	.36	....	....	....	....
	Def.	*.14*	*.20*	*.27*	*.35*	*.45*	*.55*	*.67*	*.80*	*.93*	*1.1*	*1.2*	*1.4*	....	....	....	....

The figures give the safe uniform load in tons, based on extreme fiber stress of 16,000 lb., or the end reactions from safe uniform load in thousands of pounds.

For load concentrated at center, use one-half of figures given for safe loads and four-fifths of the values given for deflections.

Figures for deflections are given in inches.

For figures at right of heavy zigzag lines, deflections are considered excessive for plastered ceilings.

TABLE 16.

Maximum Moments and Web Resistances.

Carnegie Channels.

Depth of Channel.	Weight per Foot.	Thickness of Web.	Maximum Bending Moment.	Web Resistance. Web Shear.	Web Resistance. Minimum Span.	Web Resistance. Web Buckling.	Minimum End Bearing	End Reaction, a = 3½".
d		t	M_{max}	V		f_b	a	R
In.	Lb.	In.	Ft.-Lb.	Lb.	Ft.	Lb. per Sq. In.	In.	Lb.
15	55.0	.814	76 270	122 100	2.50	15 810	5.74	93 290
	50.0	.716	71 420	107 400	2.66	15 370	6.01	79 790
	45.0	.618	66 470	92 700	2.87	14 800	6.39	66 290
	40.0	.520	61 570	78 000	3.16	14 000	6.96	52 790
	35.0	.422	56 670	63 300	3.58	12 840	7.93	39 290
	33.9	.400	55 570	60 000	3.70	12 510	8.24	36 270
13	50.0	.787	64 190	102 310	2.51	16 140	4.81	85 730
	45.0	.673	59 910	87 490	2.74	15 660	5.05	71 120
	40.0	.560	55 660	72 800	3.06	14 980	5.43	56 620
	37.0	.492	53 110	63 960	3.32	14 420	5.76	47 900
	35.0	.447	51 420	58 110	3.54	13 960	6.06	42 120
	31.8	.375	48 720	48 750	4.00	13 000	6.75	32 900
12	40.0	.755	43 670	90 600	1.93	16 250	4.39	79 740
	35.0	.632	39 730	75 840	2.10	15 710	4.64	64 540
	30.0	.510	35 830	61 200	2.34	14 920	5.04	49 470
	25.0	.387	31 890	46 440	2.75	13 630	5.81	34 280
	20.7	.280	28 470	33 600	3.39	11 570	7.37	21 060
10	35.0	.820	30 720	82 000	1.50	16 890	3.42	83 090
	30.0	.673	27 460	67 300	1.63	16 430	3.59	66 330
	25.0	.526	24 190	52 600	1.84	15 710	3.87	49 570
	20.0	.379	20 920	37 900	2.21	14 430	4.43	32 810
	15.3	.240	17 830	24 000	2.94	11 790	5.98	16 970
9	25.0	.612	20 900	55 080	1.52	16 450	3.22	57 900
	20.0	.448	17 950	40 320	1.78	15 520	3.55	39 980
	15.0	.285	15 010	25 650	2.34	13 530	4.40	22 180
	13.4	.230	14 020	20 700	2.71	12 220	5.11	16 160
8	21.25	.579	15 870	46 320	1.37	16 610	2.82	52 880
	18.75	.487	14 570	38 960	1.50	16 160	2.95	43 270
	16.25	.395	13 260	31 600	1.68	15 490	3.16	33 650
	13.75	.303	11 950	24 240	1.97	14 430	3.55	24 040
	11.5	.220	10 770	17 600	2.45	12 700	4.30	15 370
7	19.75	.629	12 590	44 030	1.14	17 070	2.35	56 380
	17.25	.524	11 450	36 680	1.25	16 690	2.44	45 910
	14.75	.419	10 310	29 330	1.21	16 110	2.60	35 430
	12.25	.314	9 170	21 980	1.67	15 140	2.87	24 950
	9.8	.210	8 030	14 700	2.19	13 220	3.54	14 580
6	15.5	.559	8 650	33 540	1.03	17 140	2.00	47 910
	13.0	.437	7 670	26 220	1.17	16 620	2.11	36 320
	10.5	.314	6 690	18 840	1.42	15 690	2.32	24 630
	8.2	.200	5 780	12 000	1.67	13 800	2.85	13 800
5	11.5	.472	5 520	23 600	0.94	17 170	1.66	38 490
	9.0	.325	4 710	16 250	1.16	16 340	1.81	25 220
	6.7	.190	3 950	9 500	1.67	14 440	2.21	13 030
4	7.25	.320	3 030	12 800	0.95	16 840	1.38	24 240
	6.25	.247	2 770	9 880	1.12	16 200	1.47	18 000
	5.4	.180	2 530	7 200	1.40	15 150	1.64	12 270
3	6.0	.356	1 830	10 680	0.68	17 540	0.96	26 540
	5.0	.258	1 630	7 740	c.84	16 990	1.02	18 630
	4.1	.170	1 450	5 100	1.14	15 950	1.13	11 520

See Table 10.

TABLE 17

Safe Loads, in Tons, and Deflections, Carnegie Channels

American Bridge Company Standards

Size	Weight per Foot, Pounds	Length of Span in Feet															
		8	9	10	11	12	13	14	15	16	18	20	22	24	26	28	30
15″	55.	38	34	31	28	25	24	22	20	19	17	15	14	13	12	11	10
	50.	36	32	29	26	24	22	20	19	18	16	14	13	12	11	10	9.5
	45.	33	30	27	24	22	21	19	18	17	15	13	12	11	10	9.5	8.9
	40.	31	27	25	22	21	19	18	16	15	14	12	11	10	9.5	8.8	8.2
	35.	28	25	23	21	19	18	16	15	14	13	11	10	9.5	8.8	8.1	7.6
	33.9	28	25	22	20	19	17	16	15	14	12	11	10	9.3	8.6	7.9	7.4
	Def.	*.07*	*.09*	*.11*	*.13*	*.16*	*.19*	*.22*	*.25*	*.28*	*.36*	*.44*	*.53*	*.64*	*.75*	*.87*	*.99*
12″	40.	22	19	18	16	15	13	13	12	11	9.7	8.8	8.0	7.3	6.7	6.3	5.8
	35.	20	18	16	14	13	12	11	10	10	8.9	8.0	7.2	6.6	6.1	5.7	5.3
	30.	18	16	14	13	12	11	10	9.6	9.0	8.0	7.2	6.5	6.0	5.5	5.1	4.8
	25.	16	14	13	12	11	9.9	9.1	8.5	8.0	7.1	6.4	5.8	5.3	4.9	4.6	4.3
	20.7	14	13	11	10	9.5	8.8	8.1	7.6	7.1	6.3	5.7	5.2	4.7	4.4	4.1	3.8
	Def.	*.09*	*.11*	*.14*	*.17*	*.20*	*.23*	*.27*	*.31*	*.35*	*.45*	*.55*	*.67*	*.79*	*.93*	*1.1*	*1.2*
10″	35.	15	14	12	11	10	9.5	8.8	8.2	7.7	6.8	6.2	5.6	5.1	4.7	4.4	4.1
	30.	14	12	11	10	9.2	8.5	7.9	7.3	6.9	6.1	5.5	5.0	4.6	4.2	3.9	3.7
	25.	12	11	9.7	8.8	8.1	7.5	6.9	6.5	6.1	5.4	4.9	4.4	4.0	3.7	3.5	3.2
	20.	11	9.3	8.4	7.6	7.0	6.5	6.0	5.6	5.3	4.7	4.2	3.8	3.5	3.2	3.0	2.8
	15.3	8.9	7.9	7.1	6.5	5.9	5.5	5.1	4.8	4.5	4.0	3.6	3.2	3.0	2.7	2.6	2.4
	Def.	*.11*	*.13*	*.17*	*.20*	*.24*	*.28*	*.32*	*.37*	*.42*	*.54*	*.66*	*.80*	*.95*	*1.1*	*1.3*	*1.5*
9″	25.	10	9.3	8.4	7.6	7.0	6.4	6.0	5.6	5.2	4.7	4.2	3.8	3.5	3.2	3.0	2.8
	20	9.0	8.0	7.2	6.6	6.0	5.5	5.1	4.8	4.5	4.0	3.6	3.3	3.0	2.8	2.6	2.4
	15.	7.5	6.7	6.0	5.5	5.0	4.6	4.3	4.0	3.8	3.3	3.0	2.7	2.5	2.3	2.2	2.0
	13.4	7.0	6.2	5.6	5.1	4.7	4.3	4.0	3.7	3.5	3.1	2.8	2.6	2.3	2.2	2.0	1.9
	Def.	*.12*	*.15*	*.18*	*.22*	*.27*	*.31*	*.36*	*.41*	*.47*	*.60*	*.74*	*.89*	*1.1*	*1.2*	*1.4*	*1.7*

The figures give the safe uniform load in tons, based on extreme fiber stress of 16,000 lb., or the end reactions from safe uniform load in thousands of pounds.

For load concentrated at center, use one-half of figures given for safe loads and four-fifths of the values given for deflections.

Figures for deflections are given in inches.

For figures at right of heavy zigzag lines, deflections are considered excessive for plastered ceilings.

TABLE 17.—*Continued*

Safe Loads, in Tons, and Deflections, Carnegie Channels
American Bridge Company Standards

Size	Weight per Foot, Pounds	Length of Span in Feet															
		5	6	7	8	9	10	11	12	13	14	15	16	18	20	22	24
8″	21.25	13	11	9.1	7.9	7.1	6.4	5.8	5.3	4.9	4.6	4.2	4.0	....	....	....	....
	18.75	12	9.7	8.4	7.3	6.5	5.8	5.3	4.9	4.5	4.2	3.9	3.7	....	....	....	....
	16.25	11	8.9	7.6	6.7	5.9	5.3	4.8	4.4	4.1	3.8	3.5	3.3	....	....	....	....
	13.75	9.6	8.0	6.9	6.0	5.3	4.8	4.4	4.0	3.7	3.4	3.2	3.0	....	....	....	....
	11.5	8.6	7.2	6.2	5.4	4.8	4.3	3.9	3.6	3.3	3.1	2.9	2.7	....	....	....	....
	Def.	*.05*	*.07*	*.10*	*.13*	*.17*	*.21*	*.25*	*.30*	*.35*	*.41*	*.47*	*.53*	....	....	....	....
7″	19.75	10.	8.4	7.2	6.3	5.6	5.1	4.6	4.2	3.9	3.6	3.4	3.2	....	....	....	....
	17.25	9.2	7.7	6.6	5.8	5.1	4.6	4.2	3.8	3.5	3.3	3.1	2.9	....	....	....	....
	14.75	8.3	6.9	5.9	5.2	4.6	4.1	3.8	3.5	3.2	3.0	2.8	2.6	....	....	....	....
	12.25	7.4	6.1	5.3	4.6	4.1	3.7	3.4	3.1	2.8	2.6	2.5	2.3	....	....	....	....
	9.8	6.7	5.6	4.8	4.2	3.7	3.3	3.0	2.8	2.6	2.4	2.2	2.1	....	....	....	....
	Def.	*.06*	*.09*	*.12*	*.15*	*.19*	*.24*	*.29*	*.34*	*.40*	*.46*	*.53*	*.61*	....	....	....	....
6″	15.5	7.0	5.8	5.0	4.3	3.9	3.5	3.2	2.9	2.7	2.5	2.3	2.2	....	....	....	....
	13.	6.2	5.1	4.4	3.9	3.4	3.1	2.8	2.6	2.4	2.2	2.1	1.9	....	....	....	....
	10.5	5.4	4.5	3.8	3.4	3.0	2.7	2.4	2.2	2.1	1.9	1.8	1.7	....	....	....	....
	8.2	4.6	3.9	3.3	2.9	2.6	2.3	2.1	1.9	1.8	1.7	1.5	1.4	....	....	....	....
	Def.	*.07*	*.10*	*.14*	*.18*	*.22*	*.28*	*.33*	*.40*	*.47*	*.54*	*.62*	*.71*	....	....	....	....
5″	11.5	4.4	3.7	3.2	2.8	2.5	2.2	2.0	1.9	1.7	1.6	1.5	1.4	....	....	....	....
	9.	3.8	3.2	2.7	2.4	2.1	1.9	1.7	1.6	1.5	1.4	1.3	1.2	....	....	....	....
	6.7	3.2	2.6	2.3	2.0	1.8	1.6	1.4	1.3	1.2	1.1	1.0	.99	....	....	....	....
	Def.	*.08*	*.12*	*.16*	*.21*	*.27*	*.33*	*.40*	*.48*	*.56*	*.65*	*.74*	*.85*	....	....	....	....
4″	7.25	2.4	2.0	1.7	1.5	1.4	1.2	1.1	1.0	.94	.87	.81	.76	....	....	....	....
	6.25	2.2	1.9	1.6	1.4	1.2	1.1	1.0	.93	.86	.80	.74	.70	....	....	....	....
	5.4	2.0	1.7	1.4	1.3	1.1	1.0	.92	.84	.78	.72	.67	.63	....	....	....	....
	Def.	*.10*	*.15*	*.20*	*.26*	*.34*	*.41*	*.50*	*.60*	*.70*	*.81*	*.93*	*1.1*	....	....	....	....
3″	6.	1.5	1.2	1.1	.92	.82	.74	.67	.61	.57	.53	.49	.46	....	....	....	....
	5.	1.3	1.1	.94	.82	.73	.66	.60	.55	.50	.47	.44	.41	....	....	....	....
	4.1	1.2	.97	.83	.73	.64	.58	.53	.48	.45	.41	.39	.36	....	....	....	....
	Def.	*.14*	*.20*	*.27*	*.35*	*.45*	*.55*	*.67*	*.80*	*.93*	*1.1*	*1.2*	*1.4*	....	....	....	....

The figures give the safe uniform load in tons, based on extreme fiber stress of 16,000 lb., or the end reactions from safe uniform load in thousands of pounds.

For load concentrated at center, use one-half of figures given for safe loads and four-fifths of the values given for deflections.

Figures for deflections are given in inches.

For figures at right of heavy zigzag lines, deflections are considered excessive for plastered ceilings.

TABLE 18.

Safe Loads, in Tons, and Deflections, Carnegie Channels Laid Flat.
American Bridge Company Standards.

Size	Weight per Foot, Pounds.	Length of Span in Feet. 3	4	5	6	7	8	9
15″	55.	7.2	5.4	4.3	3.6	3.1	2.7	2.4
	50.	6.8	5.1	4.1	3.4	2.9	2.6	2.3
	45.	6.4	4.8	3.9	3.2	2.8	2.4	2.1
	40.	5.9	4.5	3.6	3.0	2.5	2.2	2.0
	35.	5.7	4.3	3.4	2.8	2.4	2.1	1.9
	33.9	5.6	4.2	3.4	2.8	2.4	2.1	1.9
	Def.	*.03*	*.05*	*.08*	*.12*	*.16*	*.21*	*.26*
12″	40.	4.4	3.3	2.6	2.2	1.9	1.6	1.5
	35.	4.0	3.0	2.4	2.0	1.7	1.5	1.3
	30.	3.7	2.8	2.2	1.8	1.6	1.4	1.2
	25.	3.4	2.5	2.0	1.7	1.4	1.3	1.1
	20.7	3.1	2.3	1.9	1.5	1.3	1.2	1.0
	Def.	*.03*	*.06*	*.09*	*.14*	*.18*	*.24*	*.30*
10″	35.	3.3	2.5	2.0	1.6	1.4	1.2	1.1
	30.	2.9	2.2	1.7	1.4	1.2	1.1	1.0
	25.	2.7	2.0	1.6	1.3	1.1	1.0	.89
	20.	2.4	1.8	1.4	1.2	1.0	.89	.79
	15.3	2.1	1.5	1.2	1.0	.89	.78	.69
	Def.	*.04*	*.07*	*.11*	*.15*	*.21*	*.27*	*.34*
9″	25.	2.4	1.8	1.4	1.2	1.0	.90	.80
	20.	2.1	1.6	1.3	1.0	.90	.79	.70
	15.	1.8	1.3	1.1	.91	.78	.68	.61
	13.4	1.7	1.3	1.0	.86	.74	.65	.57
	Def.	*.04*	*.08*	*.12*	*.17*	*.22*	*.29*	*.37*

Size.	Weight per Foot, Pounds.	Length of Span in Feet. 3	4	5	6	7	8	9
8″	21.25	1.9	1.5	1.2	.98	.84	.74	.65
	18.75	1.8	1.3	1.1	.91	.78	.68	.61
	16.25	1.7	1.2	1.0	.84	.72	.63	.56
	13.75	1.5	1.1	.92	.77	.66	.58	.51
	11.5	1.4	1.0	.84	.70	.60	.53	.47
	Def.	*.05*	*.08*	*.13*	*.18*	*.24*	*.32*	*.40*
7″	19.75	1.7	1.3	1.0	.85	.73	.64	.57
	17.25	1.5	1.1	.93	.77	.66	.58	.52
	14.75	1.4	1.0	.84	.70	.60	.53	.47
	12.25	1.2	.95	.76	.63	.54	.47	.42
	9.8	1.1	.85	.67	.56	.48	.42	.37
	Def.	*.05*	*.09*	*.14*	*.20*	*.26*	*.35*	*.44*
6″	15.5	1.3	.98	.78	.65	.56	.49	.43
	13.	1.1	.87	.69	.58	.50	.43	.39
	10.5	1.0	.76	.61	.51	.43	.38	.34
	8.2	.88	.66	.53	.44	.38	.33	.29
	Def.	*.05*	*.10*	*.15*	*.22*	*.29*	*.38*	*.48*
5″	11.5	.95	.71	.57	.47	.41	.36	.32
	9.	.81	.60	.48	.40	.35	.30	.27
	6.7	.67	.50	.40	.34	.29	.25	.22
	Def.	*.06*	*.11*	*.17*	*.24*	*.32*	*.42*	*.54*

The figures give the safe uniform load in tons, based on extreme fiber stress of 16,000 lb., or the end reactions from safe uniform load in thousands of pounds.

For load concentrated at center, use one-half of figures given for safe loads and four-fifths of the values given for deflections. Figures for deflections are given in inches.

For figures at right of heavy zigzag lines, deflections are excessive for plastered ceilings.

TABLE 18A.

Coefficients of Deflection, Uniformly Distributed Loads.

For Concentrated Load at center use four-fifths the tabular coefficient.

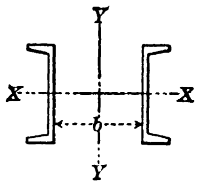
Span, Feet.	Fiber Stress, Pounds per Square Inch. 16000	14000	12500	Span, Feet.	Fiber Stress, Pounds per Square Inch. 16000	14000	12500	Span, Feet.	Fiber Stress, Pounds per Square Inch. 16000	14000	12500
1	0.017	0.014	0.013	16	4.237	3.708	3.310	31	15.906	13.918	12.427
2	0.066	0.058	0.052	17	4.783	4.186	3.737	32	16.949	14.830	13.241
3	0.149	0.130	0.116	18	5.363	4.692	4.190	33	18.025	15.772	14.082
4	0.265	0.232	0.207	19	5.975	5.228	4.668	34	19.134	16.742	14.948
5	0.414	0.362	0.323	20	6.621	5.793	5.172	35	20.276	17.741	15.841
6	0.596	0.521	0.466	21	7.299	6.387	5.703	36	21.451	18.770	16.759
7	0.811	0.710	0.634	22	8.011	7.010	6.259	37	22.659	19.827	17.703
8	1.059	0.927	0.828	23	8.756	7.661	6.841	38	23.901	20.913	18.672
9	1.341	1.173	1.047	24	9.534	8.342	7.448	39	25.175	22.028	19.668
10	1.655	1.448	1.293	25	10.345	9.052	8.082	40	26.483	23.172	20.690
11	2.003	1.752	1.565	26	11.189	9.790	8.741	41	27.824	24.346	21.737
12	2.383	2.086	1.862	27	12.066	10.558	9.427	42	29.197	25.548	22.810
13	2.797	2.448	2.185	28	12.977	11.354	10.138	43	30.603	26.779	23.909
14	3.244	2.839	2.534	29	13.920	12.180	10.875	44	31.954	28.039	25.034
15	3.724	3.259	2.909	30	14.897	13.034	11.638	45	33.517	29.328	26.185

To find the deflection in inches of a section symmetrical about the neutral axis, such as beams, channels, zees, etc., divide the coefficient in the table corresponding to given span and fiber stress by the depth of the section in inches. For unsymmetrical sections, such as angles and channels laid flat, divide the coefficient by twice the distance from neutral axis to most extreme fiber.

TABLE 19.

MOMENTS OF INERTIA OF TWO CHANNELS, BOTH AXES.
FLANGES TURNED OUT, DISTANCES FROM BACK TO BACK.

Properties of Two Channels, Flanges Turned Out.



For Distances Measured from Back to Back.

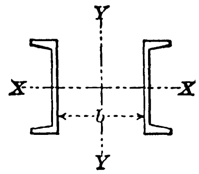
Depth.	5″		6″		7″		8″			9″		
Weight.	6.7	9.00	8.2	10.50	9.8	12.25	11.5	13.75	16.25	13.4	15.00	20.00
Area 2[s	3.90	5.26	4.76	6.14	5.70	7.16	6.70	8.04	9.52	7.78	8.78	11.72
I_x–2[s	14.8	17.6	26.0	30.2	42.2	48.2	64.6	71.6	79.6	94.6	101.4	121.2
Flange 2[s	3½	3¾	4	4¼	4¼	4½	4½	4¾	5	5	5	5¼
b	Moments of Inertia of 2 Channels About Axis Y–Y for Various Distances Back to Back. In.⁴.											
3″	16.4	22.1	20.8	26.5	25.8	32.0	31.5	37.3	44.0	38.1	42.4	56.0
3¼	18.4	24.8	23.2	29.7	28.8	35.8	35.1	41.6	49.0	42.3	47.2	62.3
3½	20.5	27.7	25.9	33.1	32.0	39.7	38.9	46.1	54.4	46.8	52.2	69.0
3¾	22.8	30.7	28.6	36.6	35.4	44.0	42.9	50.9	60.1	51.5	57.5	76.1
4	25.1	33.9	31.6	40.4	38.9	48.4	47.1	55.9	66.0	56.4	63.1	83.5
4¼	27.6	37.3	34.6	44.4	42.6	53.1	51.6	61.2	72.3	61.6	68.9	91.3
4½	30.2	40.8	37.8	48.6	46.5	58.0	56.2	66.8	78.9	67.1	75.0	99.4
4¾	33.0	44.5	41.2	52.9	50.6	63.1	61.0	72.6	85.7	72.8	81.4	107.9
5	35.8	48.4	44.7	57.5	54.8	68.4	66.1	78.6	92.9	78.7	88.1	116.8
5¼	38.8	52.4	48.4	62.2	59.2	74.0	71.3	84.9	100.3	84.9	95.1	126.1
5½	41.9	56.6	52.2	67.2	63.8	79.8	76.8	91.5	108.1	91.3	102.3	135.7
5¾	45.1	61.0	56.2	72.3	68.6	85.8	82.5	98.2	116.1	97.9	109.8	145.7
6	48.4	65.5	60.3	77.6	73.6	92.0	88.4	105.3	124.5	104.8	117.6	156.0
6¼	51.9	70.2	64.6	83.1	78.7	98.5	94.5	112.6	133.2	112.0	125.6	166.8
6½	55.5	75.1	69.0	88.8	84.0	105.2	100.8	120.2	142.1	119.3	133.9	177.8
6¾	59.2	80.1	73.5	94.8	89.5	112.1	107.3	128.0	151.4	127.0	142.5	189.3
7	63.0	85.1	78.2	100.8	95.2	119.2	114.0	136.1	160.9	134.8	151.4	201.1
7¼	67.0	90.5	83.1	107.1	101.0	126.6	120.9	144.4	170.8	143.0	160.6	213.3
7½	71.1	96.0	88.1	113.6	107.1	134.2	128.1	153.0	180.9	151.3	170.0	225.9
7¾	75.3	101.7	93.3	120.3	113.3	142.0	135.4	161.8	191.3	160.0	179.7	238.8
8	79.6	107.5	98.6	127.2	119.6	150.1	143.0	170.9	202.0	168.8	189.7	252.1
8¼	84.0	113.5	104.0	134.2	126.2	158.3	150.8	180.2	213.0	177.8	200.0	265.8
8½	88.6	119.7	109.6	141.5	132.9	166.8	158.7	189.8	224.4	187.2	210.5	279.8
8¾	93.3	126.1	115.4	148.9	139.9	175.5	166.9	200.0	236.0	196.7	221.3	294.2
9	98.1	132.6	121.3	156.6	146.9	184.4	175.3	209.7	247.9	206.5	232.4	309.0
9¼	103.0	139.3	127.3	164.4	154.2	193.6	183.9	220.1	260.2	216.6	243.7	324.1
9½	108.0	146.1	133.5	172.5	161.7	203.0	192.8	230.7	272.7	227.0	255.3	339.6
9¾	113.2	153.1	140.0	180.7	169.3	212.6	201.8	241.5	285.6	235.7	267.2	355.5
10	118.5	160.3	146.4	189.1	177.1	222.4	211.0	252.6	298.7	248.2	279.4	371.7
10¼	123.9	167.7	153.0	197.7	185.1	232.5	220.5	264.0	312.1	259.3	291.9	388.3
10½	129.5	175.2	159.8	206.5	193.3	242.8	230.1	275.6	325.8	270.5	304.6	405.3
10¾	135.1	182.8	166.7	215.5	201.6	253.3	240.0	287.4	339.9	282.1	317.6	422.6
11	140.9	190.7	173.8	224.7	210.1	264.1	250.1	300.0	354.2	293.8	330.9	440.3
11¼	146.8	198.7	181.1	234.1	218.8	275.0	260.3	311.9	368.8	305.1	344.5	458.4
11½	152.8	206.8	188.4	243.6	227.7	286.2	270.8	324.5	383.8	317.9	358.3	476.9
11¾	159.0	215.2	196.0	253.4	236.7	297.6	281.5	337.4	399.0	330.3	372.4	495.7
12	165.3	223.7	203.7	263.4	246.0	309.3	292.4	350.5	414.5	343.0	386.8	514.8

TABLE 19.—*Continued.*

Moments of Inertia of Two Channels, Both Axes.

Flanges Turned Out, Distances from Back to Back.

Properties of Two Channels, Flanges Turned Out.



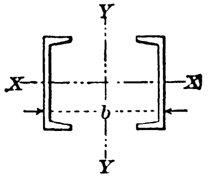
For Distances Measured from Back to Back.

Depth.	10″			12″				15″					
Weight.	15.30	20.00	25.00	20.7	25.00	30.00	35.00	33.90	35.00	40.00	45.00	50.00	55.00
Area 2[s	8.94	11.72	14.66	12.06	14.64	17.58	20.52	19.80	20.46	23.40	26.34	29.28	32.22
I_x–2[s	133.8	157.0	181.4	256.2	287.0	322.4	357.6	625.2	637.4	692.6	747.8	802.8	858.0
Flange 2[s	5¼	5½	5¾	6	6¼	6½	6¾	6¾	7	7	7¼	7½	7¾
b	Moments of Inertia of 2 Channels About Axis Y-Y for Various Distances Back to Back. In.⁴												
5″	92.5	119.4	149.9	131.6	157.5	188.5	221.8	231.3	239.6	272.3	306.9	343.4	381.7
5¼	99.6	128.7	161.6	141.5	169.4	202.8	238.5	247.9	256.8	292.0	329.0	368.2	409.1
5½	107.0	138.4	1 3.8	151.7	181.8	217.6	255.9	265.1	274.7	312.4	352.0	393.8	437.5
5¾	114.7	148.5	186.4	162.3	194.6	233.0	273.9	283.0	293.2	333.5	375.9	420.4	466.9
6	122.7	158.9	199.4	173.3	207.9	248.9	292.6	301.5	312.4	355.4	400.5	447.9	497.3
6¼	131.0	169.7	213.0	184.6	221.7	265.4	311.9	320.6	332.2	378.0	426.0	476.4	528.8
6½	139.5	180.9	227.0	196.4	235.9	282.5	331.9	340.3	352.7	401.3	452.3	505.7	561.2
6¾	148.3	192.4	241.4	208.5	250.5	300.0	352.5	360.6	373.8	425.0	479.5	536.0	594.7
7	157.4	204.3	256.3	221.0	265.7	318.2	373.8	381.5	395.5	450.2	507.5	567.2	629.1
7¼	166.8	216.6	271.7	233.8	281.2	336.9	395.7	403.1	417.9	475.8	536.2	599.3	664.6
7½	176.4	229.2	287.5	247.1	297.3	356.1	418.2	425.3	440.9	502.1	565.9	632.3	701.1
7¾	186.3	242.2	303.8	260.7	313.8	375.9	441.4	448.1	464.6	529.1	596.3	666.2	738.6
8	196.6	255.5	320.6	274.7	330.8	396.3	465.3	471.5	489.0	556.9	627.6	701.1	777.1
8¼	207.0	269.2	337.8	2 9.1	348.2	417.2	489.7	495.5	5 3.9	585.3	659.7	736.9	816.6
8½	217.8	283.3	355.5	303.8	366.1	438.6	514.8	520.2	539.5	614.6	692.6	77 .6	857.2
8¾	228.8	297.8	3 3.6	318.9	384.4	4 0.6	540.6	545.	565.8	644.5	726.4	811.2	898.7
9	240.2	312.7	392.2	334.4	403.2	483.2	567.0	571.4	592.7	675.2	761.0	849.8	941.3
9¼	251.7	327.9	4 1.2	350.3	422.5	506.3	5 4.0	597.9	620.3	706.7	796.4	889.2	984.9
9½	263.6	343.4	430.7	366.6	442.2	530.0	621.7	625.0	648.5	738.8	832.7	929.6	1029.4
9¾	275.8	359.4	450.7	383.2	462.4	554.2	650.0	652.8	677.3	771.7	869.8	970.9	1075.0
10	288.2	375.7	471.1	400.2	483.0	578.9	679.0	681.2	7 6.7	805.4	907.7	1013.2	1121.6
10¼	300.9	392.3	492.0	417.6	504.1	604.2	708.6	710.1	736.9	839.7	946.4	1056.4	1169.2
10½	313.9	409.4	513.	435.	525.6	630.1	738.8	739.7	767.6	874.8	958.9	1100.4	1217.9
10¾	327.2	426.8	535.2	453.5	547.7	656.5	769.8	770.0	799.0	910.7	1026.3	1145.4	1267.5
11	340.7	444.6	557.4	472.0	570.1	683.5	801.4	800.8	830.9	947.3	1067.6	1191.2	1318.1
11¼	354.6	462.7	580.1	490.9	593.1	711.0	833.6	832.3	863.6	984.6	1109.6	1238.1	1369.8
11½	368.7	481.2	603.3	510.2	616.5	739.1	866.4	864.4	896.9	1022.6	1152.5	1285.8	1422.5
11¾	383.1	500.1	627.0	539.9	640.3	767.7	899.9	897.1	930.9	1061.4	1196.2	1334.4	1476.2
12	397.7	519.4	651.0	549.8	664.6	796.8	934.0	930.4	965.5	1100.9	1240.7	1384.0	1530.9
12¼	412.7	539.0	675.5	570.2	689.4	826.6	968.7	964.3	1000.7	1141.2	1286.0	1434.5	1586.6
12½	427.9	558.9	700.6	591.0	714.6	856.8	1004.1	998.9	1036.6	1182.2	1332.2	1485.9	1643.3
12¾	443.4	579.3	726.0	612.1	740.3	887.7	1040.2	1034.1	1073.2	1223.9	1379.2	1538.2	1701.0
13	459.2	600.0	752.0	633.7	766.5	919.0	1076.9	1069.9	1110.4	1266.3	1427.1	1591.5	1759.7
13¼	475.2	621.1	778.4	655.6	793.1	951.0	1114.2	1106.3	1148.2	1309.5	1475.7	1645.7	1819.5
13½	491.6	642.5	805.2	677.9	820.2	983.4	1152.2	1143.3	1186.6	1353.5	1525.2	1700.8	1880.2
13¾	508.2	664.4	832.6	700.6	847.7	1016.5	1190.8	1181.0	1225.7	1398.1	1575.5	1756.8	1942.0
14	525.1	686.6	860.3	723.6	875.7	1050.1	1230.1	1219.2	1265.5	1443.5	1626.7	1813.7	2004.8
14¼	542.3	709.1	888.6	747.0	904.1	1084.2	1270.0	1258.1	1305.9	1489.7	1678.6	1871.6	2068.6
14½	559.7	732.0	917.3	770.8	933.1	1118.9	1310.5	1297.6	1347.0	1536.5	1731.4	1930.4	2133.4
14¾	577.4	755.3	946.4	795.0	962.4	1154.1	1351.7	1337.8	1388.6	1584.1	1785.0	1990.0	2199.2
15	595.5	789.0	976.0	819.5	992.3	1189.9	1393.6	1378.5	1431.0	1632.4	1839.5	2050.7	2266.0
15¼	613.7	803.0	1006.1	844.5	1022.5	1226.2	1436.0	1419.9	1473.9	1681.5	1894.8	2112.2	2333.9
15½	632.3	827.4	1036.7	869.8	1053.3	1263.1	1479.2	1461.9	1517.5	1731.3	1950.9	2174.6	2402.7
15¾	651.2	852.1	1067.6	895.5	1084.5	1300.5	1522.9	1504.5	1561.8	1781.9	2007.8	2238.0	2472.6
16	670.4	877.3	1099.1	921.5	1116.1	1338.5	1567.3	1547.7	1606.7	1833.1	2065.6	2302.3	2543.5

TABLE 20.

Moments of Inertia of Two Channels, Both Axes.
Flanges Turned In, Distances from Back to Back.

Properties of Two Channels, Flanges Turned in.



For Distances Measured from Back to Back.

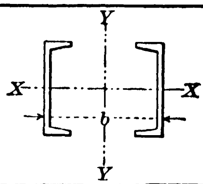
Depth.	7″		8″			9″			10″		
Weight.	9.80	12.25	11.5	13.75	16.25	13.4	15.00	20.00	15.3	20.00	25.00
Area 2[s	5.70	7.16	6.72	8.04	9.52	7.78	8.78	11.72	8.94	11.72	14.66
I_x–2[s	42.2	48.2	64.6	71.6	79.6	94.6	101.4	121.2	133.8	157.0	181.4
Web 2[s	7/16	5/8	7/16	5/8	13/16	7/16	9/16	7/8	1/2	3/4	1 1/16
b	Moments of Inertia of 2 Channels about Axis Y-Y for Various Distances Back to Back. In.^{4}.										
7″	51.7	66.0	59.9	73.1	86.4	68.6	78.6	104.8	77.6	104.0	128.7
7¼	56.0	71.4	64.9	79.2	93.6	74.4	85.1	113.6	84.1	112.7	139.5
7½	60.5	77.1	70.2	85.5	101.1	80.4	92.0	122.7	90.9	121.7	150.8
7¾	65.1	83.0	75.6	92.1	108.9	86.6	99.1	132.2	98.0	131.1	162.5
8	70.0	89.2	81.2	98.9	117.0	93.1	106.5	142.1	105.4	140.9	174.7
8¼	75.0	95.5	87.0	106.0	125.3	99.8	114.1	152.3	113.0	151.1	187.4
8½	80.2	102.1	93.1	113.3	134.0	106.8	122.0	162.9	120.9	161.6	200.5
8¾	85.5	108.9	99.4	120.9	143.0	114.0	130.3	173.8	129.1	172.5	214.1
9	91.1	116.0	105.8	128.7	152.3	121.4	138.7	185.2	137.6	183.7	228.1
9¼	96.8	123.2	112.5	136.8	161.8	129.1	147.5	196.8	146.3	195.4	242.6
9½	102.7	130.7	119.4	145.2	171.7	137.1	156.5	208.9	155.3	207.4	257.5
9¾	108.8	138.4	126.5	153.8	181.9	145.3	165.8	221.3	164.7	219.7	272.9
10	115.0	146.4	133.8	162.6	192.4	153.7	175.4	234.1	174.2	232.4	288.8
10¼	121.5	154.5	141.3	171.7	203.1	162.3	185.3	247.3	184.1	245.5	305.1
10½	128.1	162.9	149.0	181.1	214.2	171.2	195.4	260.8	194.2	259.0	321.9
10¾	134.9	171.5	157.0	190.7	225.6	180.4	205.8	274.7	204.7	272.8	339.2
11	141.9	180.4	165.1	200.5	237.2	189.8	216.5	289.0	215.4	287.0	356.9
11¼	149.0	189.4	173.5	210.6	249.2	199.4	227.5	303.6	226.3	301.6	375.0
11½	156.3	198.7	182.0	221.0	261.5	209.3	238.7	318.6	237.6	316.5	393.7
11¾	163.8	208.2	190.8	231.6	274.1	219.4	250.2	334.0	249.1	331.8	412.7
12	171.5	218.0	199.8	242.5	286.9	229.8	262.0	349.7	261.0	347.5	432.3
12¼	179.4	227.9	209.0	253.6	300.1	240.4	274.1	365.8	273.1	363.5	452.3
12½	187.4	238.1	218.4	265.0	313.6	251.3	286.4	382.3	285.4	379.9	472.8
12¾	195.6	248.5	228.0	276.6	327.3	262.4	299.1	399.2	298.1	396.7	493.7
13	204.0	259.2	237.8	288.5	341.3	273.7	312.0	416.4	311.0	413.8	515.1
13¼	212.6	270.0	247.8	300.6	355.7	285.3	325.1	433.9	324.2	431.3	536.9
13½	221.4	281.1	258.1	313.0	370.3	297.1	338.6	451.9	337.7	449.2	559.2
13¾	230.3	292.4	268.5	325.6	385.3	309.2	352.3	470.2	351.5	467.4	582.0
14	239.4	304.0	279.1	338.5	400.5	321.5	366.3	488.9	365.5	486.0	605.2
14¼	248.7	315.7	289.9	351.7	416.1	334.0	380.6	507.9	379.8	505.0	628.9
14½	258.1	327.7	301.0	365.1	43[illegible].0	346.8	395.1	527.3	390.5	524.4	653.0
14¾	267.8	339.9	312.3	378.7	448.1	359.9	409.9	547.0	409.3	544.1	677.6
15	277.6	352.4	323.8	392.6	464.5	373.2	425.0	567.2	424.5	564.1	702.6
15¼	287.6	365.0	335.5	406.8	481.3	386.7	440.4	587.7	439.9	584.6	728.1
15½	297.8	377.9	347.4	421.2	498.3	400.5	456.0	608.6	455.7	605.4	754.1
15¾	308.1	391.0	359.5	435.8	515.7	414.5	472.0	629.9	471.7	626.6	780.5
16	318.7	404.4	371.9	450.7	533.3	428.8	488.2	651.5	487.9	648.1	807.4
16¼	329.4	417.9	384.4	465.9	551.3	443.3	504.7	673.5	504.5	670.0	834.8
16½	340.3	431.7	397.2	481.3	569.5	458.0	521.4	695.8	521.3	692.3	862.6
16¾	351.3	445.7	410.1	497.0	588.1	473.0	538.4	718.6	538.4	715.0	890.9
17	362.6	460.0	423.3	512.9	606.9	488.2	555.8	741.6	555.8	738.0	919.6
17¼	374.0	474.4	436.6	529.1	626.0	503.7	573.3	765.1	573.5	761.3	948.8
17½	385.6	489.1	450.2	545.5	645.5	519.4	591.2	788.9	591.4	785.1	978.4
17¾	397.4	504.0	464.0	562.2	665.2	535.3	609.3	813.1	609.7	809.2	1008.5
18	409.3	519.2	478.0	579.1	685.2	551.6	627.7	837.6	628.2	833.7	1039.1

TABLE 20.—*Continued.*

MOMENTS OF INERTIA OF TWO CHANNELS, BOTH AXES.

FLANGES TURNED IN, DISTANCES FROM BACK TO BACK.

Properties of Two Channels, Flanges Turned In.



For Distances Measured from Back to Back.

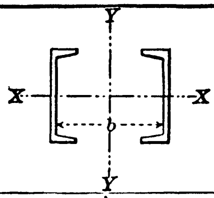
Depth.	12″					15″					
Weight.	20.7	25	30	35	40	33.9	35	40	45	50	55
Area 2[s	12.06	14.64	17.58	20.52	23.46	19.80	20.46	23.40	26.34	29.28	32.22
I_X–2[s	256.2	287.0	322.4	357.6	393.0	625.2	637.4	692.6	747.8	802.8	858.0
Web 2[s	$\frac{9}{16}$	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{9}{16}$	$\frac{13}{16}$	$\frac{7}{8}$	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{7}{16}$	$1\frac{5}{8}$
b	Moments of Inertia of 2 Channels About Axis Y–Y for Various Distances Back to Back. In.⁴.										
9″	181.6	223.8	268.2	309.9	349.0	288.[illegible]	300.4	343.7	385.5	424.6	461.9
9¼	193.2	238.1	285.4	329.8	371.6	307.1	319.8	366.0	410.4	452.2	492.1
9½	205.2	252.8	303.0	350.4	394.9	326.3	339.9	388.9	436.3	480.8	523.4
9¾	217.6	268.0	321.3	371.6	418.9	346.2	360.6	412.6	462.9	510.3	555.7
10	230.4	283.7	340.1	393.4	443.7	366.7	381.9	437.0	490.4	540.7	589.0
10¼	243.5	299.8	359.4	415.9	469.2	387.9	403.9	462.2	518.7	572.0	623.3
10½	257.1	316.3	379.3	439.0	495.5	409.6	426.5	488.1	547.8	604 2	658.6
10¾	270.9	333.3	399.7	462.7	522.5	432.0	449.8	514.7	577.8	637.4	694.9
11	285.2	350.9	420.7	487.2	550.2	455.0	473.7	542.1	608.5	671.5	732.2
11¼	299.9	368.8	442.3	512.2	578.7	478.6	498.3	570.2	640.2	706.5	770.6
11½	314.9	387.2	464.4	537.9	607.9	502.8	523.5	599.0	672.6	742.4	809.9
11¾	330.3	406.0	487.0	564.2	637.9	527.7	549.3	628.6	705 9	779 3	850.3
12	346.1	425.4	510.2	591.2	668.5	553.1	575.8	658.9	739.9	817.0	891.7
12¼	362.2	445.1	534.0	618.8	699.9	579.2	602.9	690.0	774.9	855.7	934.1
12½	378.8	465.4	558.3	647.1	732.0	605.9	630.7	721.7	810.6	895.3	977.5
12¾	395.7	486.1	583.1	676.0	764.9	633.2	659.1	754.3	847.2	935.8	1021.9
13	413.0	507.3	608.5	705.6	798.5	661.1	688.2	787.5	884.6	977.3	10 7.3
13¼	430.6	528.9	634.5	735.8	832.8	689.7	717.9	821.5	922.8	1019.6	1113.8
13½	448.7	551.0	661.0	766.6	867.9	718.9	748.2	856.2	961.9	1062.9	1161.2
13¾	467.1	573.6	688.0	798.1	903.7	748.7	779.2	891.7	1001.8	1107.1	1209.7
14	485.9	596.6	715.7	830.2	940.3	779.1	810.8	927.9	1042.5	1152.3	1259.1
14¼	505.0	620.1	743.8	863.0	977.6	810.1	843.1	964.8	1084.0	1198.3	1309.6
14½	524.6	644.0	772.5	896.4	1015.6	841.7	876.0	1002.4	1126.4	1245.2	1361.1
14¾	544.5	666.4	801.8	930.4	1054.3	874.0	909.6	1040.8	1169.6	1293.1	1413.6
15	564.8	693.2	831.6	965.1	1093.8	906.9	943.8	1080.0	1213.6	1341.9	1467.1
15¼	585.5	718.5	862.0	1000.5	1134.0	940.4	978.7	1119.8	1258.4	1391.7	1521.7
15½	606.6	744 3	892.9	1036.5	1175.0	974.5	1014.2	1160.4	1304.1	1442.3	1577.2
15¾	628.0	770.5	924.4	1073.1	1216.7	1009.3	1050.3	1201.7	1350.6	1493.9	1633.7
16	649.8	797.2	956.4	1110.3	1259.1	1044.6	1087.1	1243.8	1397.9	1546.3	1691.3
16¼	672.0	824.3	989.0	1148.2	1302.3	1080.6	1124.5	1286.6	1446.1	1599.7	1749.9
16½	694.5	851.9	1022.1	1186.8	1346.2	1117.2	1162.6	1330.2	1495.1	1654.0	1809.4
16¾	717.5	879.9	1055.8	1226.0	1390.8	1154.4	1201.3	1374.4	1544.9	1709.3	1870.0
17	740.8	908.5	1090.0	1265.8	1436.2	1192.2	1240.6	1419.4	1595.5	1765.4	1931.7
17¼	764.5	937.4	1124.8	1306.3	1482.3	1230.7	1280.6	1465.2	1647.0	1822.5	1994.3
17½	788.6	966.9	1160.1	1347.4	1529.1	1269.8	1321.3	1511.7	1699.3	1880.5	2057.9
17¾	813.0	996.8	1196.0	1389.2	1576.7	1309.5	1362.5	1558.9	1752.5	1939.4	2122.5
18	837.8	1027.1	1232.4	1431.6	1625.0	1349.8	1404.5	1606.8	1806.4	1999.2	2188.2
18¼	863.0	1057.9	12[illegible]9.4	1474.7	1674.0	1 90.7	1447.0	1655.5	1861.2	2060.0	2254.8
18½	888.6	1089.2	1306.9	1518.4	1723.8	1432.3	1490.2	1704.9	1916.8	2121.6	2322.5
18¾	914.6	1120.9	1345.0	1562.8	1774.3	1474.4	1534.1	1755.1	1973.3	2184.2	2391.2
19	940.9	1153.1	1383.6	1607.7	1825.6	1517.2	1578.6	1806.0	2030.5	2247.7	2460.8
19¼	967.6	1185.8	1422.8	1653.3	1877.5	1560.6	1623.7	1857.6	2088.6	2312.2	2531.6
19½	994.7	1218.9	1462.5	1699.6	1930.3	1604.6	1669.5	1910.0	2147.5	2377.5	2603.3
19¾	1022.2	1252.4	1502.8	1746.5	1983.7	1649.3	1715.9	1963.1	2207.3	2443.8	2676.0
20	1050 0	1286.5	1543.6	1794.1	2037.9	1694.5	1763.0	2016.9	2267.8	2510.9	2749.8

TABLE 21.

Moments of Inertia of Two Channels, Both Axes.

Flanges Turned In, Distances Inside to Inside of Web.

Properties of Two Channels, Flanges Turned In.



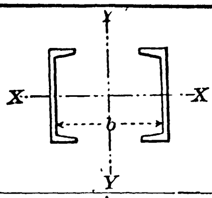
For Distances Measured from Inside to Inside of Web.

Depth.	7		8			9			10		
Weight.	9.80	12.25	11.50	13.75	16.25	13.4	15.00	20.00	15.3	20.00	25.00
Area 2[s	5.70	7.16	6.70	8.04	9.52	7.78	8.78	11.72	8.92	11.72	14.66
I_x—2[s	42.2	48.2	64.6	71.6	79.6	94.6	101.4	121.2	133.8	157.0	181.4
Web 2[s	$\frac{7}{16}$	$\frac{5}{8}$	$\frac{7}{16}$	$\frac{5}{8}$	$\frac{13}{16}$	$\frac{7}{16}$	$\frac{9}{16}$	$\frac{7}{8}$	$\frac{1}{2}$	$\frac{3}{4}$	$1\frac{1}{16}$
b	Moments of Inertia of 2 Channels About Axis Y–Y for Various Distances Inside to Inside of Web. In.^{4}.										
7″	59.1	80.4	68.9	88.6	110.5	79.4	94.2	138.1	90.4	131.5	177.7
$7\frac{1}{4}$	63.7	86.4	74.3	95.3	118.6	85.6	101.4	148.1	97.4	141.3	190.5
$7\frac{1}{2}$	68.4	92.7	79.8	102.2	127.0	92.1	108.9	158.6	104.8	151.5	203.7
$7\frac{3}{4}$	73.4	99.2	85.7	109.5	135.8	98.7	116.6	169.4	112.4	162.0	217.4
8	78.5	105.9	91.6	116.9	144.8	105.7	124.6	180.6	120.3	172.9	231.5
$8\frac{1}{4}$	83.8	112.8	97.8	124.6	154.2	112.8	132.9	192.1	128.4	184.2	246.1
$8\frac{1}{2}$	89.3	120.0	104.3	132.6	163.8	120.2	141.5	204.0	136.9	195.8	261.2
$8\frac{3}{4}$	95.0	127.4	110.9	140.8	173.7	127.9	150.3	216.3	145.6	207.8	276.7
9	100.8	135.0	117.7	149.2	184.0	135.8	159.5	229.0	154.6	220.2	292.7
$9\frac{1}{4}$	106.9	142.8	124.8	158.0	194.5	143.9	168.9	242.0	163.9	233.0	309.1
$9\frac{1}{2}$	113.1	150.9	132.0	166.9	205.3	152.3	178.5	255.4	173.5	246.1	326.0
$9\frac{3}{4}$	119.4	159.2	139.5	176.2	216.5	160.9	188.5	269.1	183.3	259.5	343.4
10	126.0	167.7	147.2	185.6	227.9	169.8	198.7	283.2	193.4	273.4	361.2
$10\frac{1}{4}$	132.7	176.4	155.1	195.4	239.6	178.9	209.2	297.7	203.8	287.6	379.5
$10\frac{1}{2}$	139.6	185.4	163.1	205.3	251.6	188.3	220.0	312.6	214.5	302.2	398.2
$10\frac{3}{4}$	146.7	194.6	171.5	215.8	264.0	197.9	231.1	327.8	225.5	317.1	417.4
11	154.0	204.0	180.0	226.1	276.6	207.7	242.4	343.4	236.7	332.4	437.0
$11\frac{1}{4}$	161.5	213.6	188.7	236.8	289.5	217.8	254.0	359.4	248.2	348.1	457.2
$11\frac{1}{2}$	169.1	223.5	197.6	247.8	302.7	228.1	265.9	375.7	260.0	364.2	477.7
$11\frac{3}{4}$	176.9	233.6	206.8	259.0	316.3	238.7	278.0	392.4	272.1	380.6	498.7
12	184.9	243.9	216.1	270.5	330.1	249.5	290.4	409.4	284.4	397.4	520.2
$12\frac{1}{4}$	193.0	254.5	225.7	282.3	344.2	260.6	303.2	426.9	297.1	414.5	542.2
$12\frac{1}{2}$	201.4	265.2	235.4	294.3	358.6	271.9	316.2	444.7	310.0	432.0	564.6
$12\frac{3}{4}$	209.9	276.2	245.4	306.5	373.3	283.4	329.4	462.8	323.2	449.9	587.5
13	218.6	287.4	255.6	319.0	388.3	295.2	342.9	481.3	336.6	468.2	610.8
$13\frac{1}{4}$	227.4	298.9	266.0	331.8	403.6	307.3	356.7	500.2	350.4	486.8	634.6
$13\frac{1}{2}$	236.5	310.5	276.6	344.8	419.2	319.5	370.8	519.5	364.4	505.8	658.8
$13\frac{3}{4}$	245.7	322.4	287.4	358.1	435.1	332.0	385.2	539.1	378.7	525.1	683.5
14	255.1	334.5	298.4	371.6	451.4	344.8	399.8	559.1	393.3	544.8	708.7
$14\frac{1}{4}$	264.7	346.9	309.7	385.3	467.9	357.8	414.7	579.5	408.1	564.9	734.3
$14\frac{1}{2}$	274.5	359.4	321.1	399.4	484.7	371.1	429.9	600.2	423.3	585.4	760.4
$14\frac{3}{4}$	284.4	372.2	332.8	413.6	501.8	384.5	445.4	621.3	438.7	606.2	786.9
15	294.5	385.2	344.6	428.2	519.2	398.3	461.1	642.8	454.4	627.4	813.9
$15\frac{1}{4}$	304.8	398.5	356.7	442.9	536.9	412.3	477.1	664.6	470.4	649.0	841.4
$15\frac{1}{2}$	315.3	411.9	369.0	458.0	554.9	426.5	493.4	686.9	486.6	670.9	869.3
$15\frac{3}{4}$	326.0	425.6	381.5	473.2	573.2	440.9	510.0	709.4	503.1	693.2	897.7
16	336.8	439.5	394.2	488.8	591.8	455.6	526.8	732.4	519.9	715.9	926.5
$16\frac{1}{4}$	347.8	453.7	407.1	504.6	610.7	470.6	543.9	755.7	537.0	738.9	955.8
$16\frac{1}{2}$	359.0	468.0	420.2	520.6	629.9	485.8	561.3	779.3	554.4	762.3	985.6
$16\frac{3}{4}$	370.4	482.6	433.5	536.9	649.4	501.2	579.0	803.4	572.0	786.1	1015.8
17	381.9	497.4	447.1	553.4	669.1	516.9	596.9	827.8	590.0	810.2	1046.5
$17\frac{1}{4}$	393.6	512.5	460.8	570.2	689.3	532.8	615.2	852.6	608.2	834.7	1077.6
$17\frac{1}{2}$	405.5	527.7	474.8	587.3	709.6	549.0	633.6	877.7	626.7	859.6	1109.2
$17\frac{3}{4}$	417.6	543.2	488.9	604.6	730.3	565.4	652.4	903.2	645.4	884.8	1141.3
18	429.9	558.9	503.3	622.1	751.3	582.0	671.5	929.1	664.5	910.4	1173.8

TABLE 21.—*Continued.*

Moments of Inertia of Two Channels, Both Axes.

Flanges Turned In, Distances Inside to Inside of Web.



Properties of Two Channels, Flanges Turned in.

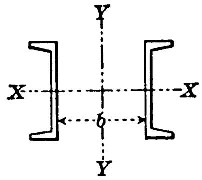
For Distances Measured from Inside to Inside of Web.

Depth.	12″					15″					
Weight.	20.7	25	30	35	40	33 9	35	40	45	50	55
Area 2[s	12.06	14.64	17.58	20.52	23.46	19.80	20.46	23.40	26.34	29.28	32.22
I_x–2[s	256.2	287.0	322.4	357.6	393.0	625.2	637.4	692.6	747.8	802.8	858.0
Web 2[s	$\frac{9}{16}$	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{9}{16}$	$\frac{13}{16}$	$\frac{7}{8}$	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{7}{16}$	$1\frac{5}{8}$
b	Moments of Inertia of 2 Channels about Axis Y–Y for Various Distances Inside to Inside of Webs. In.⁴.										
9″	208.2	269.9	342.1	417.9	497.2	350.3	369.2	441.8	518.0	596.4	678.2
$9\frac{1}{4}$	220.7	285.6	361.5	441.1	524.2	370.9	390.	467.1	547.1	629.4	715.1
$9\frac{1}{2}$	233.5	301.7	381.4	464.9	552.0	392.2	413.0	493.2	577.0	663.2	753.0
$9\frac{3}{4}$	246.7	318.4	401.9	489.3	580.5	414.0	435.9	519.9	607.8	698.0	791.9
10	260.4	335.4	423.0	514.4	609.7	436.5	459.5	547.4	639.4	733.7	831.8
$10\frac{1}{4}$	274.3	353.0	444.6	540.2	639.7	459.6	483.7	575.7	671.8	770.3	872.7
$10\frac{1}{2}$	288.7	371.0	466.7	566.6	670.4	483.4	508.5	604.7	705.0	807.9	914.6
$10\frac{3}{4}$	303.4	389.5	498.4	593.6	701.9	507.7	533.9	634.4	739.1	846.4	957.6
11	318.6	408.4	512.7	621.3	734.1	532.7	560.0	664.8	774.0	885.7	1001.5
$11\frac{1}{4}$	334.0	427.8	536.5	649.6	767.0	558.3	586.8	696.0	809.7	926.0	1046.5
$11\frac{1}{2}$	350.0	447.6	560.9	678.6	800.6	584.5	614.2	727.9	846.3	967.3	1092.4
$11\frac{3}{4}$	366.1	467.9	585.8	708.2	835.0	611.3	642.2	760.6	883.6	1009.4	1139.4
12	382.8	488.7	611.2	738.4	870.2	638.7	670.9	794.0	921.8	1052.5	1187.4
$12\frac{1}{4}$	399.8	510.0	637.2	769.3	906.0	666.8	700.2	828.1	960.9	1096.4	1236.4
$12\frac{1}{2}$	417.2	531.6	663.8	800.9	942.6	695.5	730.2	862.9	1000.7	1141.3	1286.4
$12\frac{3}{4}$	434.9	553.7	690.9	833.0	979.9	724.8	760.8	898.5	1041.4	1187.2	1337.5
13	453.0	576.3	718.6	865.9	1018.0	754.7	792.0	934.9	1082.9	1233.9	1389.5
$13\frac{1}{4}$	471.6	599.4	746.8	899.3	1056.8	785.2	824.0	971.9	1125.3	1281.5	1442.5
$13\frac{1}{2}$	490.4	622.9	775.6	933.4	1096.3	816.4	856.5	1009.7	1168.5	1330.1	1496.6
$13\frac{3}{4}$	509.7	646.9	804.9	968.2	1136.6	848.2	889.7	1048.2	1212.5	1379.6	1551.7
14	529.3	671.3	834.8	1003.6	1177.6	880.5	923.5	1087.6	1257.3	1430.0	1607.8
$14\frac{1}{4}$	549.4	696.2	865.2	1039.6	1219.4	913.5	958.0	1127.6	1302.9	1481.4	1664.9
$14\frac{1}{2}$	569.7	721.6	896.2	1076.3	1261.8	947.2	993.1	1168.3	1349.4	1533.6	1723.0
$14\frac{3}{4}$	590.5	747.4	927.6	1113.6	1305.0	981.4	1028.9	1209.8	1396.7	1586.8	1782.1
15	611.7	773.6	959.8	1151.6	1349.0	1016.3	1065.2	1252.0	1444.9	1640.9	1842.2
$15\frac{1}{4}$	633.2	800.4	992.4	1190.2	1393.7	1051.8	1102.3	1294.9	1493.8	1695.9	1903.4
$15\frac{1}{2}$	655.1	827.6	1025.6	1229.5	1439.1	1087.9	1140.0	1338.6	1543.6	1751.9	1965.5
$15\frac{3}{4}$	677.4	855.2	1059.3	1269.3	1485.2	1124.6	1178.3	1383.0	1594.3	1808.7	2028.7
16	700.0	883.3	1093.6	1309.9	1532.1	1161.9	1217.3	1428.2	1645.7	1866.5	2092.8
$16\frac{1}{4}$	723.0	911.9	1128.4	1351.1	1579.7	1199.9	1256.9	1474.1	1698.0	1925.1	2158.0
$16\frac{1}{2}$	746.5	940.9	1163.8	1392.9	1628.0	1238.5	1297.1	1520.7	1751.1	1984.8	2224.2
$16\frac{3}{4}$	770.2	970.4	1199.7	1435.4	1677.1	1277.7	1338.0	1568.0	1805.0	2045.3	2291.4
17	794.4	1000.4	1236.2	1478.5	1727.0	1317.5	1379.6	1616.1	1859.8	2106.7	2359.6
$17\frac{1}{4}$	818.9	1030.8	1273.2	1522.2	1777.6	1357.9	1421.8	1664.9	1915.3	2169.1	2428.9
$17\frac{1}{2}$	843.9	1061.7	1310.8	1566.6	1828.9	1399.0	1464.6	1714.5	1971.8	2232.4	2499.1
$17\frac{3}{4}$	869.1	1093.0	1349.0	1611.7	1880.9	1440.6	1508.1	1764.8	2029.0	2296.6	2570.4
18	894.8	1124.8	1387.7	1657.4	1933.6	1482.9	1552.2	1815.8	2087.1	2361.7	2642.6
$18\frac{1}{4}$	920.9	1157.0	1426.9	1703.7	1987.1	1525.8	1596.9	1867.6	2146.0	2427.8	2715.9
$18\frac{1}{2}$	947.3	1189.7	1466.7	1750.7	2041.4	1569.4	1642.3	1920.1	2205.7	2494.7	2790.2
$18\frac{3}{4}$	974.1	1222.9	1507.0	1798.3	2096.3	1613.5	1688.4	1973.3	2266.2	2562.6	2865.5
19	1001.3	1256.5	1547.9	1846.5	2152.1	1658.3	1735.1	2027.3	2327.6	2631.4	2941.8
$19\frac{1}{4}$	1028.8	1290.6	1589.4	1895.4	2208.5	1703.7	1782.4	2082.0	2389.8	2701.1	3019.1
$19\frac{1}{2}$	1056.8	1325.1	1631.4	1945.0	2265.7	1749.7	1830.4	2137.4	2452.8	2771.8	3097.5
$19\frac{3}{4}$	1085.1	1360.1	1673.9	1995.2	2323.6	1796.3	1880.0	2193.6	2516.7	2843.3	3176.8
20	1113.7	1395.6	1717.0	2046.0	2382.2	1843.5	1928.3	2250.5	2581.4	2915.8	3257.1

TABLE 22.

Properties of Two Channels, Spaced Small Distances.

Properties of Two Channels. Flanges Turned Out



For Distances Measured from Back to Back.

Channels.		Total Area.	Axis X-X.		Axis Y-Y.									
					b = 0.		b = ¼″.		b = ½″.		b = ¾″.		b = 2″.	
Depth.	Weight.		I_x	r_x	I_y	r_y	I_y	r_y	I_y	r_y	I_y	r_y	I_y	r_y
In.	Lb.	In.²	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.⁴	In.
	4.1	2.38	3.2	1.17									5.4	1.50
3	5	2.92	3.6	1.12	1.1	0.60	1.4	0.70					6.6	1.50
	6	3.50	4.2	1.08	1.4	0.62	1.8	0.71	2.4	0.82	3.1	0.93	8.1	1.52
	5.4	3.12	7.6	1.56	1.3	0.65	1.7	0.74	2.2	0.84	2.8	0.95	7.3	1.53
4	6¼	3.64	8.2	1.51	1.6	0.64	2.0	0.73	2.6	0.84	3.4	0.95	8.5	1.52
	7¼	4.24	9.0	1.46	1.8	0.65	2.4	0.74	3.0	0.84	3.9	0.95	10.0	1.53
5	6.7	3.90	14.8	1.95	1.9	0.69	2.4	0.78	3.1	0.89	3.9	0.99	9.6	1.57
	9	5.26	17.6	1.83	2.5	0.68	3.2	0.78	4.1	0.88	5.2	0.98	12.9	1.56
	8.2	4.78	26.0	2.34	2.7	0.74	3.4	0.84	4.2	0.93	5.2	1.03	12.4	1.61
6	10½	6.14	30.2	2.21	3.3	0.73	4.2	0.82	5.3	0.92	6.5	1.02	15.7	1.60
	13	7.62	34.6	2.13	4.2	0.74	5.3	0.83	6.6	0.93	8.2	1.03	19.7	1.61
	9.8	5.70	42.2	2.72	3.7	0.80	4.5	0.89	5.6	0.99	6.8	1.09	15.6	1.65
7	12¼	7.16	48.2	2.59	4.4	0.78	5.5	0.87	6.7	0.97	8.3	1.07	19.2	1.63
	14¾	8.64	54.2	2.50	5.3	0.78	6.6	0.87	8.1	0.97	10.0	1.07	23.3	1.64
	11.5	6.72	64.6	3.11	4.9	0.85	6.0	0.94	7.2	1.03	8.7	1.14	19.3	1.70
8	13¾	8.04	71.6	2.98	5.6	0.83	6.8	0.92	8.3	1.01	10.1	1.12	22.7	1.68
	16¼	9.52	79.6	2.89	6.5	0.83	8.0	0.91	9.8	1.01	11.8	1.11	26.7	1.67
	13.4	7.78	94.6	3.49	6.4	0.90	7.7	0.99	9.3	1.09	11.0	1.19	23.6	1.74
9	15	8.78	101.4	3.40	7.0	0.89	8.4	0.97	10.1	1.07	12.1	1.12	26.2	1.72
	20	11.72	121.2	3.21	8.9	0.87	10.0	0.96	13.1	1.05	15.7	1.15	34.5	1.71
	15.3	8.94	133.8	3.87	8.2	0.96	9.8	1.05	11.6	1.14	13.7	1.24	28.6	1.79
	20	11.72	157.0	3.66	10.0	0.92	12.0	1.01	14.3	1.10	17.0	1.20	36.2	1.75
10	25	14.66	181.4	3.52	12.4	0.92	14.9	1.00	17.9	1.10	21.3	1.20	45.4	1.76
	30	17.60	206.0	3.42	15.2	0.93	18.4	1.02	22.1	1.12	26.3	1.22	55.9	1.78
	35	20.54	230.4	3.35	19.2	0.96	23.1	1.06	27.6	1.16	32.8	1.26	68.5	1.82
	20.7	12.06	256.2	4.61	13.4	1.05	16.1	1.15	18.8	1.24	21.9	1.34	42.8	1.89
	25	14.64	287.0	4.43	15.8	1.03	18.5	1.12	21.7	1.21	25.3	1.31	50.5	1.85
12	30	17.58	322.4	4.28	18.5	1.02	21.7	1.11	25.5	1.20	29.9	1.30	60.0	1.85
	35	20.52	357.6	4.17	21.7	1.02	25.5	1.11	30.1	1.21	35.3	1.31	70.9	1.86
	40	23.46	393.0	4.09	25.5	1.04	30.1	1.13	35.4	1.22	41.5	1.32	83.0	1.88
	33.9	19.80	623.2	5.62	28.8	1.20	33.1	1.29	38.0	1.38	43.5	1.48	80.2	2.01
	35	20.46	637.4	5.58	29.8	1.20	34.1	1.28	39.1	1.38	44.8	1.47	82.8	2.01
	40	23.40	692.6	5.43	33.1	1.18	38.1	1.27	43.8	1.36	50.3	1.46	93.5	1.99
15	45	26.34	747.8	5.32	37.1	1.18	42.6	1.26	49.1	1.36	56.3	1.45	105.2	1.99
	50	29.28	802.8	5.23	41.2	1.18	47.7	1.27	55.0	1.36	63.2	1.46	118.1	2.00
	55	32.22	858.0	5.16	46.1	1.19	53.2	1.28	61.4	1.37	70.5	1.47	131.9	2.02

TABLE 23

Properties of Equal Leg Angles

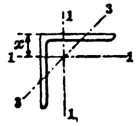
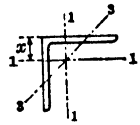
Size of Angle	Thickness	Weight per Foot	Area	Distance from Center of Gravity to Back of Angle	 Moment of Inertia	Section Modulus	Radius of Gyration	Least Radius of Gyration Axis 3–3	Maximum Bending Moment @ 16,000 Lb. per Sq. In. Axis 1–1
				x	I_1	S_1	r_1	r_3	M_1
Inches	Inches	Pounds	Inches²	Inches	Inches⁴	Inches³	Inches	Inches	Foot-Pounds
8×8	$1\frac{1}{4}$	62.7	18.44	2.45	106.56	19.21	2.40	1.55	25 600
	$1\frac{3}{16}$	59.8	17.59	2.43	102.31	18.38	2.41	1.55	24 500
	$1\frac{1}{8}$	56.9	16.73	2.41	97.97	17.53	2.42	1.55	23 400
	$1\frac{1}{16}$	54.0	15.87	2.39	93.53	16.67	2.43	1.56	22 200
	1	51.0	15.00	2.37	88.98	15.80	2.44	1.56	21 100
	$\frac{15}{16}$	48.1	14.12	2.34	84.33	14.92	2.44	1.56	19 900
	$\frac{7}{8}$	45.0	13.23	2.32	79.58	14.02	2.45	1.57	18 700
	$\frac{13}{16}$	42.0	12.34	2.30	74.72	13.11	2.46	1.57	17 500
	$\frac{3}{4}$	38.9	11.44	2.28	69.74	12.19	2.47	1.57	16 200
	$\frac{11}{16}$	35.8	10.53	2.25	64.64	11.25	2.48	1.58	15 000
	$\frac{5}{8}$	32.7	9.61	2.23	59.43	10.30	2.49	1.58	13 700
	$\frac{9}{16}$	29.6	8.68	2.21	54.09	9.34	2.50	1.58	12 500
	$\frac{1}{2}$	26.4	7.75	2.19	48.63	8.37	2.50	1.58	11 200
6×6	1	37.4	11.00	1.86	35.46	8.57	1.80	1.16	11 400
	$\frac{15}{16}$	35.3	10.37	1.84	33.72	8.11	1.80	1.16	10 800
	$\frac{7}{8}$	33.1	9.73	1.82	31.92	7.63	1.81	1.17	10 200
	$\frac{13}{16}$	31.0	9.09	1.80	30.06	7.15	1.82	1.17	9 550
	$\frac{3}{4}$	28.7	8.44	1.78	28.15	6.66	1.83	1.17	8 900
	$\frac{11}{16}$	26.5	7.78	1.75	26.19	6.17	1.83	1.17	8 250
	$\frac{5}{8}$	24.2	7.11	1.73	24.16	5.66	1.84	1.18	7 550
	$\frac{9}{16}$	21.9	6.43	1.71	22.07	5.14	1.85	1.18	6 850
	$\frac{1}{2}$	19.6	5.75	1.68	19.91	4.61	1.86	1.18	6 150
	$\frac{7}{16}$	17.2	5.06	1.66	17.68	4.07	1.87	1.19	5 450
	$\frac{3}{8}$	14.9	4.36	1.64	15.39	3.53	1.88	1.19	4 700
5×5	1	30.6	9.00	1.61	19.64	5.80	1.48	.96	7 730
	$\frac{15}{16}$	28.9	8.50	1.59	18.71	5.49	1.48	.96	7 320
	$\frac{7}{8}$	27.2	7.98	1.57	17.75	5.17	1.49	.96	6 890
	$\frac{13}{16}$	25.4	7.47	1.55	16.76	4.85	1.50	.97	6 470
	$\frac{3}{4}$	23.6	6.94	1.52	15.74	4.53	1.51	.97	6 040
	$\frac{11}{16}$	21.8	6.40	1.50	14.68	4.20	1.51	.97	5 600
	$\frac{5}{8}$	20.0	5.86	1.48	13.58	3.86	1.52	.97	5 150
	$\frac{9}{16}$	18.1	5.31	1.46	12.44	3.51	1.53	.98	4 680
	$\frac{1}{2}$	16.2	4.75	1.43	11.25	3.15	1.54	.98	4 200
	$\frac{7}{16}$	14.3	4.18	1.41	10.02	2.79	1.55	.98	3 720
	$\frac{3}{8}$	12.3	3.61	1.39	8.74	2.42	1.56	.99	3 230
4×4	$\frac{13}{16}$	19.9	5.84	1.29	8.14	3.01	1.18	.77	4 010
	$\frac{3}{4}$	18.5	5.44	1.27	7.67	2.81	1.19	.77	3 750
	$\frac{11}{16}$	17.1	5.03	1.25	7.17	2.61	1.19	.77	3 480
	$\frac{5}{8}$	15.7	4.61	1.23	6.66	2.40	1.20	.77	3 200
	$\frac{9}{16}$	14.3	4.18	1.21	6.12	2.19	1.21	.78	2 920
	$\frac{1}{2}$	12.8	3.75	1.18	5.56	1.97	1.22	.78	2 630
	$\frac{7}{16}$	11.3	3.31	1.16	4.97	1.75	1.23	.78	2 330
	$\frac{3}{8}$	9.8	2.86	1.14	4.36	1.52	1.23	.79	2 030
	$\frac{5}{16}$	8.2	2.40	1.12	3.72	1.29	1.24	.79	1 720
	$\frac{1}{4}$	6.6	1.94	1.09	3.04	1.05	1.25	.79	1 400

TABLE 23.—*Continued*

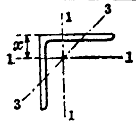
PROPERTIES OF EQUAL LEG ANGLES



Size of Angle	Thickness	Weight per Foot	Area	Distance from Center of Gravity to Back of Angle	Moment of Inertia	Section Modulus	Radius of Gyration	Least Radius of Gyration Axis 3-3	Maximum Bending Moment @ 16,000 Lb. per Sq. In. Axis 1-1
				x	I_1	S_1	r_1	r_3	M_1
Inches	Inches	Pounds	Inches²	Inches	Inches⁴	Inches³	Inches	Inches	Foot-Pounds
$3\frac{1}{2} \times 3\frac{1}{2}$	$\frac{13}{16}$	17.1	5.03	1.17	5.25	2.25	1.02	0.67	3 000
	$\frac{3}{4}$	16.0	4.69	1.15	4.96	2.11	1.03	0.67	2 810
	$\frac{11}{16}$	14.8	4.34	1.12	4.65	1.96	1.04	0.67	2 610
	$\frac{5}{8}$	13.6	3.98	1.10	4.33	1.81	1.04	0.67	2 410
	$\frac{9}{16}$	12.4	3.62	1.08	3.99	1.65	1.05	0.68	2 200
	$\frac{1}{2}$	11.1	3.25	1.06	3.63	1.49	1.06	0.68	1 990
	$\frac{7}{16}$	9.8	2.87	1.04	3.26	1.32	1.07	0.68	1 760
	$\frac{3}{8}$	8.5	2.48	1.01	2.87	1.15	1 07	0.69	1 530
	$\frac{5}{16}$	7.2	2.09	.99	2.45	.98	1.08	0.69	1 310
	$\frac{1}{4}$	5.8	1.69	.97	2.01	.79	1.09	0.69	1 050
	$\frac{3}{16}$	4.4	1.28	.94	1.55	.60	1.10	0.69	800
	$\frac{5}{32}$	3.64	1.07	.93	1.31	.51	1.10	0.69	680
3×3	$\frac{5}{8}$	11.5	3.36	.98	2.62	1.30	.88	.57	1 730
	$\frac{9}{16}$	10.4	3.06	.95	2.43	1.19	.89	.58	1 585
	$\frac{1}{2}$	9.4	2.75	.93	2.22	1.07	.90	.58	1 430
	$\frac{7}{16}$	8.3	2.43	.91	2.00	.95	.91	.58	1 270
	$\frac{3}{8}$	7.2	2.11	.89	1.76	.83	.91	.58	1 110
	$\frac{5}{16}$	6.1	1.78	.87	1.51	.71	.92	.59	950
	$\frac{1}{4}$	4.9	1.44	.84	1.24	.58	.93	.59	770
	$\frac{3}{16}$	3.71	1.09	.82	.96	.44	.94	.60	590
	$\frac{1}{8}$	2.50	0.74	.80	.66	.30	.95	.60	400
$2\frac{3}{4} \times 2\frac{3}{4}$	$\frac{1}{2}$	8.5	2.50	.87	1.67	.89	.82	.53	1 190
	$\frac{7}{16}$	7.6	2.22	.85	1.51	.79	.82	.53	1 050
	$\frac{3}{8}$	6.6	1.92	.82	1.33	.69	.83	.53	920
	$\frac{5}{16}$	5.6	1.62	.80	1.15	.59	.84	.54	790
	$\frac{1}{4}$	4.5	1.31	.78	.95	.48	.85	.54	640
	$\frac{3}{16}$	3.39	1.00	.76	.73	.37	.86	.54	490
	$\frac{1}{8}$	2.29	0.68	.73	.51	.25	.87	.55	330
$2\frac{1}{2} \times 2\frac{1}{2}$	$\frac{1}{2}$	7.7	2.25	.81	1.23	.73	.74	.47	970
	$\frac{7}{16}$	6.8	2.00	.78	1.11	.65	.74	.48	870
	$\frac{3}{8}$	5.9	1.73	.76	.98	.57	.75	.48	760
	$\frac{5}{16}$	5.0	1.47	.74	.85	.48	.76	.48	640
	$\frac{1}{4}$	4.1	1.19	.72	.70	.39	.77	.49	530
	$\frac{3}{16}$	3.07	.90	.69	.55	.30	.78	.49	400
	$\frac{1}{8}$	2.08	.61	.67	.38	.20	.79	.50	270
$2\frac{1}{4} \times 2\frac{1}{4}$	$\frac{1}{2}$	6.8	2.00	.74	.87	.58	.66	.43	770
	$\frac{7}{16}$	6.1	1.78	.72	.79	.52	.67	.43	690
	$\frac{3}{8}$	5.3	1.55	.70	.70	.45	.67	.43	600
	$\frac{5}{16}$	4.5	1.31	.68	.61	.39	.68	.44	520
	$\frac{1}{4}$	3.62	1.07	.66	.51	.32	.69	.44	430
	$\frac{3}{16}$	2.75	.81	.63	.39	.24	.70	.44	320
	$\frac{1}{8}$	1.86	.55	.61	.27	.16	.71	.45	220

TABLE 23.—*Continued*

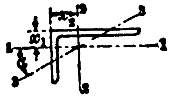
PROPERTIES OF EQUAL LEG ANGLES



Size of Angle	Thickness	Weight per Foot	Area	Distance from Center of Gravity to Back of Angle	Axis 1-1: Moment of Inertia	Axis 1-1: Section Modulus	Axis 1-1: Radius of Gyration	Least Radius of Gyration: Axis 3-3	Maximum Bending Moment @ 16,000 Lb. per Sq. In.: Axis 1-1
				x	I_1	S_1	r_1	r_3	M_1
Inches	Inches	Pounds	Inches²	Inches	Inches⁴	Inches³	Inches	Inches	Foot-Pounds
2×2	7/16	5.3	1.56	.66	.54	.40	.59	.39	530
	3/8	4.7	1.36	.64	.48	.35	.59	.39	470
	5/16	3.92	1.15	.61	.42	.30	.60	.39	400
	1/4	3.19	.94	.59	.35	.25	.61	.39	330
	3/16	2.44	.71	.57	.28	.19	.62	.40	250
	1/8	1.65	.48	.55	.19	.13	.63	.40	170
1¾×1¾	7/16	4.6	1.34	.59	.35	.30	.51	.33	400
	3/8	3.99	1.18	.57	.31	.26	.51	.34	350
	5/16	3.39	1.00	.55	.27	23	.52	.34	310
	1/4	2.77	.82	.53	.23	.19	.53	.34	250
	3/16	2.12	.63	.51	.18	.14	.54	.35	190
	1/8	1.44	.43	.48	.13	.10	.55	.35	130
1½×1½	3/8	3.35	.99	.51	.19	.19	.44	.29	250
	5/16	2.86	.84	.49	.16	.16	.44	.29	220
	1/4	2.34	.69	.47	.14	.134	.45	.29	180
	3/16	1.80	.53	.44	.11	.10	.46	.29	140
	1/8	1.23	.36	.42	.078	.072	.46	.30	90
1¼×1¼	5/16	2.33	.68	.42	.091	.109	.36	.23	150
	1/4	1.92	.56	.40	.077	.091	.37	.24	120
	3/16	1.48	.43	.38	.061	.071	.38	.24	90
	1/8	1.91	.30	.35	.044	.049	.38	.25	70
1⅛×1⅛	3/16	1.32	.39	.35	.044	.057	.34	.22	75
	1/8	.91	.27	.33	.032	.040	.34	.22	50
1×1	1/4	1.49	.44	.34	.037	.056	.29	.19	75
	3/16	1.16	.34	.32	.030	.044	.30	.19	60
	1/8	.8	.23	.30	.022	.031	.31	.20	40
	.109	.71	.21	.29	.020	.028	.31	.20	40
⅞×⅞	3/16	1.00	.30	.29	.019	.033	.26	.18	40
	1/8	.70	.21	.26	.014	.023	.26	.19	30
	3/32	.53	.16	.25	.011	.018	.27	.20	20
¾×¾	3/16	.84	.25	.26	.012	.024	.22	.15	32
	1/8	.59	.18	.23	.0088	.017	.23	.15	23
	3/32	.45	.14	.22	.0069	.013	.23	.15	17
⅝×⅝	1/8	.48	.15	.20	.0048	.0113	.18	.12	15
	3/32	.37	.11	.19	.0038	.0088	.19	.12	11
½×½	1/8	.38	.11	.17	.0023	.007	.15	.10	9
	3/32	.29	.085	.16	.0019	.0055	.15	.10	7

TABLE 24

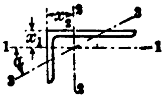
Properties of Unequal Leg Angles



Size of Angle	Thickness	Weight per Foot	Area	Distance from Center of Gravity to Back of Longer Leg	Distance from Center of Gravity to Back of Shorter Leg	Moment of Inertia Axis 1–1	Moment of Inertia Axis 2–2	Section Modulus Axis 1–1	Section Modulus Axis 2–2	Radius of Gyration Axis 1–1	Radius of Gyration Axis 2–2	Radius of Gyration Axis 3–3	Tangent of Angle a	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. Long Leg Vertical	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. Short Leg Vertical
				x_1	x_2	I_1	I_2	S_1	S_2	r_1	r_2	r_3		M_2	M_1
In.	In.	Lb.	In.²	In.	In.	In.⁴	In.⁴	In.³	In.³	In.	In.	In.		Ft.-Lb.	Ft.-Lb.
8×6	1	44.2	13.00	1.65	2.65	38.78	80.78	8.92	15.11	1.73	2.49	1.28	.543	20 150	11 900
	15/16	41.7	12.25	1.63	2.63	36.85	76.59	8.43	14.27	1.73	2.50	1.28	.545	19 030	11 250
	7/8	39.1	11.48	1.61	2.61	34.86	72.31	7.94	13.41	1.74	2.51	1.28	.546	17 900	10 600
	13/16	36.5	10.72	1.59	2.59	32.82	67.92	7.44	12.55	1.75	2.52	1.29	.549	16 730	9 900
	3/4	33.8	9.94	1.56	2.56	30.72	63.42	6.93	11.67	1.76	2.53	1.29	.553	15 560	9 250
	11/16	31.2	9.15	1.54	2.54	28.56	58.82	6.41	10.77	1.77	2.54	1.29	.556	14 400	8 550
	5/8	28.5	8.36	1.52	2.52	26.33	54.10	5.88	9.87	1.77	2.54	1.30	.554	13 160	7 850
	9/16	25.7	7.56	1.50	2.50	24.04	49.26	5.34	8.95	1.78	2.55	1.30	.556	11 930	7 100
	1/2	23.0	6.75	1.47	2.47	21.68	44.31	4.79	8.02	1.79	2.56	1.30	.558	10 700	6 400
	7/16	20.2	5.93	1.45	2.45	19.25	39.23	4.23	7.07	1.80	2.57	1.30	.560	9 420	5 640
8×3½	1	35.7	10.50	.92	3.17	7.8	66.2	3.0	13.7	.86	2.51	.73	.....	18 400	4 000
	15/16	33.7	9.90	.89	3.14	7.4	62.9	2.9	12.9	.87	2.52	.73	.....	17 200	3 870
	7/8	31.7	9.30	.87	3.12	7.1	59.4	2.7	12.2	.87	2.53	.73	.....	16 200	3 600
	13/16	29.6	8.68	.85	3.10	6.7	55.9	2.5	11.4	.88	2.54	.73	.....	15 200	3 330
	3/4	27.5	8.06	.82	3.07	6.3	52.3	2.3	10.6	.88	2.55	.73	.....	14 100	3 060
	11/16	25.3	7.43	.80	3.05	5.9	48.5	2.2	9.8	.89	2.56	.73	.....	13 000	2 930
	5/8	23.2	6.80	.78	3.03	5.4	44.7	2.0	9.0	.90	2.57	.74	.....	12 000	2 660
	9/16	21.0	6.15	.75	3.00	5.0	40.8	1.8	8.2	.90	2.57	.74	.....	10 900	2 400
	1/2	18.7	5.50	.73	2.98	4.5	36.7	1.6	7.3	.91	2.58	.74	.....	9 700	2 190
	7/16	16.5	4.84	.70	2.95	4.1	32.5	1.5	6.4	.92	2.59	.74	.....	8 600	2 000
7×3½	1	32.3	9.50	.96	2.70	7.53	45.37	2.96	10.58	.89	2.19	.74	.241	14 100	3 950
	15/16	30.5	8.97	.94	2.69	7.18	43.13	2.80	10.00	.89	2.19	.74	.244	13 350	3 740
	7/8	28.7	8.42	.91	2.66	6.83	40.82	2.64	9.42	.90	2.20	.74	.247	12 550	3 520
	13/16	26.8	7.87	.89	2.64	6.46	38.44	2.48	8.82	.91	2.21	.74	.250	11 750	3 310
	3/4	24.9	7.31	.87	2.62	6.08	35.99	2.31	8.22	.91	2.22	.74	.253	10 950	3 080
	11/16	23.0	6.75	.85	2.60	5.69	33.47	2.14	7.60	.92	2.23	.74	.257	10 150	2 850
	5/8	21.0	6.17	.82	2.57	5.28	30.87	1.97	6.97	.93	2.24	.75	.259	9 300	2 630
	9/16	19.1	5.59	.80	2.55	4.85	28.19	1.80	6.33	.93	2.25	.75	.262	8 450	2 400
	1/2	17.0	5.00	.78	2.53	4.41	25.42	1.62	5.68	.94	2.25	.75	.264	7 570	2 160
	7/16	15.0	4.40	.75	2.50	3.95	22.56	1.44	5.01	.95	2.26	.76	.267	6 680	1 920
	3/8	13.0	3.80	.73	2.48	3.48	19.60	1.26	4.33	.96	2.27	.76	.270	5 770	1 680
6×4	1	30.6	9.00	1.17	2.17	10.75	30.75	3.79	8.02	1.09	1.85	.85	.414	10 700	5 050
	15/16	28.9	8.50	1.14	2.14	10.26	29.26	3.59	7.59	1.10	1.86	.85	.418	10 120	4 790
	7/8	27.2	7.98	1.12	2.12	9.75	27.73	3.39	7.15	1.11	1.86	.86	.421	9 550	4 520
	13/16	25.4	7.47	1.10	2.10	9.23	26.15	3.18	6.70	1.11	1.87	.86	.425	8 950	4 240
	3/4	23.6	6.94	1.08	2.08	8.68	24.51	2.97	6.25	1.12	1.88	.86	.428	8 350	3 960
	11/16	21.8	6.40	1.06	2.06	8.11	22.82	2.76	5.78	1.13	1.89	.86	.431	7 700	3 680
	5/8	20.0	5.86	1.03	2.03	7.52	21.07	2.54	5.31	1.13	1.90	.86	.434	7 080	3 390
	9/16	18.1	5.31	1.01	2.01	6.91	19.26	2.31	4.83	1.14	1.90	.87	.438	6 450	3 080
	1/2	16.2	4.75	.99	1.99	6.27	17.39	2.08	4.33	1.15	1.91	.87	.440	5 770	2 770
	7/16	14.3	4.18	.96	1.96	5.60	15.46	1.85	3.83	1.16	1.92	.87	.443	5 100	2 470
	3/8	12.3	3.61	.94	1.94	4.90	13.47	1.60	3.32	1.17	1.93	.88	.446	4 430	2 140

TABLE 24.—*Continued*

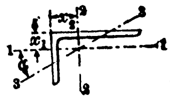
PROPERTIES OF UNEQUAL LEG ANGLES



Size of Angle	Thickness	Weight per Foot	Area	Distance from Center of Gravity to Back of Longer Leg	Distance from Center of Gravity to Back of Shorter Leg	Moment of Inertia		Section Modulus		Radius of Gyration			Tangent of Angle α	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. Long Leg Vertical	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. Short Leg Vertical
						Axis 1-1	Axis 2-2	Axis 1-1	Axis 2-2	Axis 1-1	Axis 2-2	Axis 3-3			
				x_1	x_2	I_1	I_2	S_1	S_2	r_1	r_2	r_3		M_2	M_1
In.	In.	Lb.	In.²	In.	In.	In.⁴	In.⁴	In.³	In.³	In.	In.	In.		Ft.-Lb.	Ft.-Lb.
6×3½	1	28.9	8.50	1.01	2.26	7.21	29.24	2.90	7.83	.92	1.85	.74	.317	10 450	3 870
	15/16	27.3	8.03	.99	2.24	6.88	27.84	2.74	7.41	.93	1.86	.74	.320	9 880	3 650
	7/8	25.7	7.55	.97	2.22	6.55	26.39	2.59	6.98	.93	1.87	.75	.323	9 300	3 450
	13/16	24.0	7.06	.95	2.20	6.20	24.89	2.43	6.55	.94	1.88	.75	.327	8 750	3 240
	3/4	22.4	6.56	.93	2.18	5.84	23.34	2.27	6.10	.94	1.89	.75	.331	8 150	3 030
	11/16	20.6	6.06	.90	2.15	5.47	21.74	2.11	5.65	.95	1.89	.75	.334	7 550	2 810
	5/8	18.9	5.55	.88	2.13	5.08	20.08	1.94	5.19	.96	1.90	.75	.338	6 920	2 590
	9/16	17.1	5.03	.86	2.11	4.67	18.37	1.77	4.72	.96	1.91	.75	.341	6 300	2 360
	1/2	15.3	4.50	.83	2.08	4.25	16.60	1.59	4.24	.97	1.92	.76	.344	5 650	2 120
	7/16	13.5	3.97	.81	2.06	3.81	14.77	1.41	3.75	.98	1.93	.76	.347	5 000	1 880
	3/8	11.7	3.42	.78	2.04	3.34	12.86	1.23	3.25	.99	1.94	.77	.350	4 330	1 640
	5/16	9.8	2.87	.75	2.02	2.85	10.88	1.04	2.74	1.00	1.95	.77	.353	3 650	1 380
5×4	7/8	24.2	7.11	1.21	1.71	9.23	16.45	3.31	4.99	1.14	1.52	.84	....	6 650	4 410
	13/16	22.7	6.65	1.18	1.68	8.74	15.54	3.11	4.69	1.15	1.53	.84	.....	6 250	4 150
	3/4	21.1	6.19	1.16	1.66	8.23	14.60	2.90	4.37	1.15	1.54	.84	.....	5 830	3 870
	11/16	19.5	5.72	1.14	1.64	7.70	13.62	2.69	4.05	1.16	1.54	.84	.617	5 400	3 590
	5/8	17.8	5.23	1.12	1.62	7.14	12.61	2.48	3.73	1.17	1.55	.84	.620	4 970	3 310
	9/16	16.2	4.75	1.10	1.60	6.56	11.56	2.26	3.39	1.18	1.56	.85	.623	4 520	3 010
	1/2	14.5	4.25	1.07	1.57	5.96	10.46	2.04	3.05	1.18	1.57	.85	.626	4 070	2 720
	7/16	12.8	3.75	1.05	1.55	5.33	9.32	1.81	2.70	1.19	1.58	.85	.629	3 600	2 420
	3/8	11.0	3.23	1.03	1.53	4.66	8.14	1.57	2.34	1.20	1.59	.86	.631	3 120	2 090
5×3½	7/8	22.7	6.67	1.04	1.79	6.21	15.67	2.52	4.88	.96	1.53	.75	.455	6 510	3 360
	13/16	21.3	6.25	1.02	1.77	5.89	14.81	2.37	4.58	.97	1.54	.75	.460	6 110	3 160
	3/4	19.8	5.81	1.00	1.75	5.55	13.92	2.22	4.28	.98	1.55	.75	.464	5 710	2 960
	11/16	18.3	5.37	.97	1.72	5.20	12.99	2.06	3.97	.98	1.56	.75	.468	5 290	2 750
	5/8	16.8	4.92	.95	1.70	4.83	12.03	1.90	3.65	.99	1.56	.75	.472	4 870	2 530
	9/16	15.2	4.47	.93	1.68	4.45	11.03	1.73	3.32	1.00	1.57	.75	.476	4 430	2 310
	1/2	13.6	4.00	.91	1.66	4.05	9.99	1.56	2.99	1.01	1.58	.75	.479	3 990	2 080
	7/16	12.0	3.53	.88	1.63	3.63	8.91	1.39	2.64	1.01	1.59	.76	.482	3 520	1 850
	3/8	10.4	3.05	.86	1.61	3.18	7.78	1.21	2.29	1.02	1.60	.76	.485	3 060	1 610
	5/16	8.7	2 56	.84	1.59	2.72	6.60	1.02	1.94	1.03	1.61	.76	.489	2 590	1 360
5×3	13/16	19.9	5.84	.86	1.86	3.71	13.98	1.74	4.45	.80	1.55	.64	.336	5 930	2 320
	3/4	18.5	5.44	.84	1.84	3.51	13.15	1.63	4.16	.80	1.55	.64	.340	5 550	2 170
	11/16	17.1	5.03	.82	1.82	3.29	12.28	1.51	3.86	.81	1.56	.64	.345	5 150	2 010
	5/8	15.7	4.61	.80	1.80	3.06	11.37	1.39	3.55	.82	1.57	.64	.349	4 740	1 850
	9/16	14.3	4.18	.77	1.77	2.83	10.43	1.27	3.23	.82	1.58	.65	.353	4 310	1 690
	1/2	12.8	3.75	.75	1.75	2.58	9.45	1.15	2.91	.83	1.59	.65	.357	3 880	1 530
	7/16	11.3	3.31	.73	1.73	2.32	8.43	1.02	2.58	.84	1.60	.65	.361	3 440	1 360
	3/8	9.8	2.86	.70	1.70	2.04	7.37	.89	2.24	.84	1.61	.65	.364	2 990	1 190
	5/16	8.2	2.40	.68	1.68	1.75	6.26	.75	1.89	.85	1.61	.66	.368	2 520	1 000

TABLE 24.—*Continued*

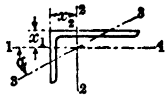
Properties of Unequal Leg Angles



Size of Angle	Thickness	Weight per Foot	Area	Distance from Center of Gravity to Back of Longer Leg	Distance from Center of Gravity to Back of Shorter Leg	Moment of Inertia		Section Modulus		Radius of Gyration			Tangent of Angle α	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. Long Leg Vertical	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. Short Leg Vertical
						Axis 1-1	Axis 2-2	Axis 1-1	Axis 2-2	Axis 1-1	Axis 2-2	Axis 3-3			
				x_1	x_2	I_1	I_2	S_1	S_2	r_1	r_2	r_3		M_2	M_1
In.	In.	Lb.	In.²	In.	In	In.⁴	In.⁴	In.³	Ins.³	In.	In.	In.¹		Ft.-Lb.	Ft.-Lb.
4½×3	13/16	18.5	5.43	.90	1.65	3.60	10.33	1.71	3.62	.81	1.38	.64	.....	4 830	2 280
	¾	17.3	5.06	.88	1.63	3.40	9.73	1.60	3.38	.82	1.39	.64	.....	4 500	2 130
	11/16	16.0	4.68	.85	1.60	3.19	9.10	1.49	3.14	.83	1.39	.64	.419	4 180	1 990
	⅝	14.7	4.30	.83	1.58	2.98	8.44	1.37	2.89	.83	1.40	.64	.424	3 850	1 830
	9/16	13.3	3.90	.81	1.56	2.75	7.75	1.25	2.64	.84	1.41	.64	.428	3 520	1 660
	½	11.9	3.50	.79	1.54	2.51	7.04	1.13	2.37	.85	1.42	.65	.431	3 160	1 510
	7/16	10.6	3.09	.76	1.51	2.25	6.29	1.01	2.10	.85	1.43	.65	.437	2 800	1 350
	⅜	9.1	2.67	.74	1.49	1.98	5.50	.88	1.83	.86	1.44	.66	.440	2 440	1 170
	5/16	7.7	2.25	.72	1.47	1.70	4.67	.75	1.54	.87	1.44	.66	.443	2 050	1 000
4×3½	13/16	18.5	5.43	1.11	1.36	5.49	7.77	2.30	2.92	1.01	1.19	.72	.....	3 900	3 070
	¾	17.3	5.06	1.09	1.34	5.18	7.32	2.15	2.74	1.01	1.20	.72	.....	3 650	2 870
	11/16	16.0	4.68	1.07	1.32	4.86	6.86	2.00	2.55	1.02	1.21	.72	.742	3 400	2 670
	⅝	14.7	4.30	1.04	1.29	4.52	6.37	1.84	2.35	1.03	1.22	.72	.742	3 140	2 460
	9/16	13.3	3.90	1.02	1.27	4.16	5.86	1.68	2.15	1.03	1.23	.72	.747	2 870	2 240
	½	11.9	3.50	1.00	1.25	3.79	5.32	1.52	1.94	1.04	1.23	.72	.750	2 590	2 030
	7/16	10.6	3.09	.98	1.23	3.40	4.76	1.35	1.72	1.05	1.24	.72	.753	2 290	1 800
	⅜	9.1	2.67	.96	1.21	2.99	4.17	1.18	1.50	1.06	1.25	.73	.755	2 000	1 570
	5/16	7.7	2.25	.93	1.18	2.56	3.56	1.00	1.26	1.07	1.26	.73	.757	1 680	1 350
4×3	13/16	17.1	5.03	.94	1.44	3.47	7.34	1.68	2.87	.83	1.21	.64	.518	3 830	2 240
	¾	16.0	4.69	.92	1.42	3.28	6.93	1.57	2.68	.84	1.22	.64	.524	3 570	2 090
	11/16	14.8	4.34	.89	1.39	3.08	6.49	1.46	2.49	.84	1.22	.64	.529	3 320	1 950
	⅝	13.6	3.98	.87	1.37	2.87	6.03	1.35	2.30	.85	1.23	.64	.534	3 070	1 800
	9/16	12.4	3.62	.85	1.35	2.66	5.55	1.23	2.10	.86	1.24	.64	.538	2 800	1 640
	½	11.1	3.25	.83	1.33	2.42	5.05	1.11	1.89	.86	1.25	.64	.543	2 520	1 490
	7/16	9.8	2.87	.80	1.30	2.18	4.52	.99	1.68	.87	1.25	.64	.547	2 240	1 320
	⅜	8.5	2.48	.78	1.28	1.92	3.96	.87	1.46	.88	1.26	.64	.551	1 950	1 160
	5/16	7.2	2.09	.76	1.26	1.65	3.38	.74	1.23	.89	1.27	.65	.554	1 640	990
	¼	5.8	1.69	.74	1.24	1.36	2.77	.60	1.00	.89	1.28	.65	.557	1 330	800
3½×3	13/16	15.8	4.62	.98	1.23	3.33	4.98	1.65	2.20	.85	1.04	.62	.694	2 930	2 200
	¾	14.7	4.31	.96	1.21	3.15	4.70	1.54	2.05	.85	1.04	.62	.698	2 730	2 050
	11/16	13.6	4.00	.94	1.19	2.96	4.41	1.44	1.91	.86	1.05	.62	.703	2 550	1 920
	⅝	12.5	3.67	.92	1.17	2.76	4.11	1.33	1.76	.87	1.06	.62	.707	2 350	1 770
	9/16	11.4	3.34	.90	1.15	2.55	3.79	1.21	1.61	.87	1.07	.62	.711	2 150	1 610
	½	10.2	3.00	.88	1.13	2.33	3.45	1.10	1.45	.88	1.07	.62	.714	1 930	1 470
	7/16	9.1	2.65	.85	1.10	2.09	3.10	.98	1.29	.89	1.08	.62	.718	1 720	1 310
	⅜	7.9	2.30	.83	1.08	1.85	2.73	.85	1.13	.90	1.09	.62	.721	1 510	1 130
	5/16	6.6	1.93	.81	1.06	1.58	2.33	.72	.96	.90	1.10	.63	.724	1 280	960
	¼	5.4	1.56	.79	1.04	1.29	1.91	.58	.78	.91	1.11	.63	.727	1 040	770
3½×2½	11/16	12.5	3.65	.77	1.27	1.72	4.13	.99	1.85	.69	1.06	.53	.468	2 470	1 320
	⅝	11.5	3.36	.75	1.25	1.61	3.85	.92	1.71	.69	1.07	.53	.472	2 280	1 230
	9/16	10.4	3.06	.73	1.23	1.49	3.55	.84	1.56	.70	1.08	.53	.480	2 080	1 120
	½	9.4	2.75	.70	1.20	1.36	3.24	.76	1.41	.70	1.09	.53	.486	1 880	1 010
	7/16	8.3	2.43	.68	1.18	1.23	2.91	.68	1.26	.71	1.09	.54	.491	1 680	910
	⅜	7.2	2.11	.66	1.16	1.09	2.56	.59	1.09	.72	1.10	.54	.496	1 450	790
	5/16	6.1	1.78	.64	1.14	.94	2.19	.50	.93	.73	1.11	.54	.501	1 240	670
	¼	4.9	1.44	.61	1.11	.78	1.80	.41	.75	.74	1.12	.54	.506	1 000	550

TABLE 24.—*Continued*

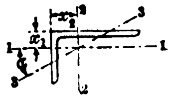
PROPERTIES OF UNEQUAL LEG ANGLES



Size of Angle	Thickness	Weight per Foot	Area	Distance from Center of Gravity to Back of Longer Leg	Distance from Center of Gravity to Back of Shorter Leg	Moment of Inertia		Section Modulus		Radius of Gyration			Tangent of Angle α	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. Long Leg Vertical	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. Short Leg Vertical
						Axis 1–1	Axis 2–2	Axis 1–1	Axis 2–2	Axis 1–1	Axis 2–2	Axis 3–3			
				x_1	x_2	I_1	I_2	S_1	S_2	r_1	r_2	r_3		M_2	M_1
In.	In.	Lb.	In.^{2}	In.	In.	In.^{4}	In.^{4}	In.^{3}	In.^{3}	In.	In.	In.		Ft.-Lb.	Ft.-Lb.
3½×2	3/8	6.6	1.93	.50	1.25	.57	2.36	.38	1.05	.54	1.11	.43	.324	1 400	500
	5/16	5.6	1.63	.48	1.23	.49	2.02	.32	.89	.55	1.12	.43	.329	1 190	430
	1/4	4.5	1.32	.46	1.21	.41	1.67	.26	.72	.56	1.13	.43	.335	960	350
3¼×2	9/16	9.0	2.64	.59	1.21	.75	2.64	.53	1.30	.53	1.00	.44	.....	1 730	700
	1/2	8.1	2.38	.57	1.19	.69	2.42	.48	1.17	.54	1.01	.44	.....	1 560	640
	7/16	7.2	2.11	.54	1.17	.62	2.18	.43	1.05	.54	1.02	.44	.....	1 400	570
	3/8	6.3	1.83	.52	1.15	.55	1.92	.37	.91	.55	1.02	.44	.....	1 210	500
	5/16	5.3	1.55	.50	1.12	.48	1.65	.32	.77	.56	1.03	.45	.....	1 020	430
	1/4	4.3	1.25	.48	1.09	.40	1.36	.26	.63	.57	1.04	.45	.369	840	350
3¼×1⅝	3/16	2.99	.88	.34	1.16	.17	.98	.13	.47	.44	1.05	.35	.....	630	170
3×2 13/16	9/16	10.1	2.96	.88	.98	2.01	2.37	1.04	1.17	.82	.90	.54	.....	1 560	1 390
3×2 11/16	9/16	9.8	2.89	.84	.99	1.76	2.38	.95	1.17	.78	.91	.55	....	1 560	1 270
3×2½	9/16	9.5	2.78	.77	1.02	1.42	2.28	.82	1.15	.72	.91	.52	.661	1 530	1 090
	1/2	8.5	2.50	.75	1.00	1.30	2.08	.74	1.04	.72	.91	.52	.666	1 390	990
	7/16	7.6	2.22	.73	.98	1.18	1.88	.66	.93	.73	.92	.52	.672	1 240	880
	3/8	6.6	1.92	.71	.96	1.04	1.66	.58	.81	.74	.93	.52	.676	1 080	770
	5/16	5.6	1.62	.68	.93	.90	1.42	.49	.69	.74	.94	.53	.680	920	650
	1/4	4.5	1.31	.66	.91	.74	1.17	.40	.56	.75	.95	.53	.684	750	530
	3/16	3.39	1.00	.64	.89	.58	.91	.31	.43	.76	.95	.53	.688	570	410
3×2	1/2	7.7	2.25	.58	1.08	.67	1.92	.47	1.00	.55	.92	.43	.414	1 330	630
	7/16	6.8	2.00	.56	1.06	.61	1.73	.42	.89	.55	.93	.43	.421	1 190	560
	3/8	5.9	1.73	.54	1.04	.54	1.53	.37	.78	.56	.94	.43	.428	1 040	490
	5/16	5.0	1.47	.52	1.02	.47	1.32	.32	.66	.57	.95	.43	.434	880	430
	1/4	4.1	1.19	.49	.99	.39	1.09	.26	.54	.57	.95	.43	.440	720	350
	3/16	3.07	.91	.47	.97	.31	.84	.20	.41	.58	.95	.43	.446	550	270
2½×2	1/2	6.8	2.00	.63	.88	.64	1.14	.46	.70	.56	.75	.42	.600	930	610
	7/16	6.1	1.78	.60	.85	.58	1.03	.41	.63	.57	.76	.42	.607	830	550
	3/8	5.3	1.55	.58	.83	.52	.91	.36	.55	.58	.77	.42	.614	730	480
	5/16	4.5	1.31	.56	.81	.45	.79	.31	.47	.58	.78	.42	.620	630	410
	1/4	3.62	1.06	.54	.79	.37	.65	.25	.38	.59	.78	.42	.626	510	330
	3/16	2.75	.81	.51	.76	.29	.51	.20	.29	.60	.79	.43	.632	390	270
2½×1¾	5/16	4.2	1.24	.47	.85	.31	.76	.24	.46	.50	.79	.37	.....	610	320
	1/4	3.40	1.00	.45	.83	.25	.62	.20	.37	.50	.79	.38	.....	500	270
	3/16	2.59	.77	.43	.81	.20	.49	.15	.29	.51	.80	.38	.....	390	200
2½×1½	5/16	3.92	1.15	.40	.90	.19	.71	.17	.44	.41	.79	.32	.349	590	230
	1/4	3.19	.94	.38	.88	.16	.59	.14	.36	.41	.79	.32	.357	480	190
	3/16	2.44	.72	.35	.85	.13	.46	.11	.28	.42	.80	.33	.364	370	150
2½×1¼	5/32	1.91	.57	.27	.89	.064	.37	.066	.23	.34	.81	.27	.265	300	90

TABLE 24.—*Continued*

Properties of Unequal Leg Angles



Size of Angle	Thickness	Weight per Foot	Area	Distance from Center of Gravity to Back of Longer Leg	Distance from Center of Gravity to Back of Shorter Leg	Moment of Inertia Axis 1-1	Moment of Inertia Axis 2-2	Section Modulus Axis 1-1	Section Modulus Axis 2-2	Radius of Gyration Axis 1-1	Radius of Gyration Axis 2-2	Radius of Gyration Axis 3-3	Tangent of Angle α	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. Long Leg Vertical	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. Short Leg Vertical
				x_1	x_2	I_1	I_2	S_1	S_2	r_1	r_2	r_3		M_2	M_1
In.	In.	Lb.	In.²	In.	In.	In.⁴	In.⁴	In.³	In.³	In.	In.	In.		Ft.-Lb.	Ft.-Lb.
2¼×1½	½	5.6	1.63	.48	.86	.26	.75	.26	.54	.40	.68	.32	.....	720	350
	7/16	5.0	1.45	.46	.83	.24	.68	.23	.48	.41	.69	.32	.....	640	300
	⅜	4.4	1.27	.44	.81	.21	.61	.20	.42	.41	.69	.32	.....	560	270
	5/16	3.66	1.07	.42	.79	.19	.53	.17	.36	.42	.70	.32	.424	480	230
	¼	2.98	.88	.39	.77	.16	.44	.14	.30	.42	.71	.32	.....	400	190
	3/16	2.28	.67	.37	.75	.12	.34	.11	.23	.43	.72	.33	.....	310	150
2×1½	⅜	3.99	1.17	.46	.71	.21	.43	.20	.34	.42	.61	.32	.524	450	270
	5/16	3.39	1.00	.44	.69	.18	.38	.17	.29	.42	.62	.32	.534	390	230
	¼	2.77	.81	.41	.66	.15	.32	.14	.24	.43	.62	.32	.543	320	190
	3/16	2.12	.62	.39	.64	.12	.25	.11	.18	.44	.63	.32	.551	240	150
	⅛	1.44	.42	.37	.62	.085	.17	.075	.13	.45	.64	.33	.559	170	100
2×1⅜	⅜	3.83	1.13	.42	.73	.16	.42	.17	.33	.38	.61	.29	.434	440	230
	5/16	3.26	.96	.39	.71	.14	.37	.14	.28	.38	.62	.29	.445	370	190
	¼	2.66	.79	.37	.68	.12	.31	.12	.23	.39	.63	.30	.455	300	160
	3/16	2.04	.60	.35	.66	.096	.24	.094	.18	.40	.63	.31	.475	240	125
2×1¼	¼	2.55	.75	.33	.71	.089	.30	.097	.23	.34	.63	.27	....	300	130
	3/16	1.96	.57	.31	.69	.071	.23	.075	.18	.35	.64	.27	.....	240	100
1¾×1¼	¼	2.34	.69	.35	.60	.085	.20	.095	.18	.35	.54	.27	.....	240	125
	3/16	1.80	.53	.33	.58	.069	.16	.075	.14	.36	.55	.27	.....	190	100
	⅛	1.23	.36	.31	.56	.049	.11	.052	.094	.37	.56	.27	.....	125	70
1¾×1⅛	¼	2.24	.66	.31	.62	.062	.19	.077	.17	.31	.54	.24	.....	230	100
	3/16	1.72	.51	.29	.60	.050	.15	.060	.13	.32	.55	.24	.....	170	80
	⅛	1.17	.35	.27	.58	.037	.11	.043	.093	.32	.56	.24	.....	125	57
1½×1¼	5/16	2.59	.76	.40	.52	.097	.16	.113	.16	.35	.45	.26	.....	210	150
	¼	2.13	.63	.38	.50	.081	.13	.093	.13	.36	.46	.26	.....	170	125
	3/16	1.64	.48	.35	.48	.065	.10	.073	.10	.37	.46	.26	.....	130	97
1⅜×1	¼	1.81	.54	.30	.49	.041	.093	.059	.106	.28	.42	.21	.....	140	80
	3/16	1.40	.41	.28	.47	.033	.075	.046	.082	.28	.43	.21	.....	110	60
	⅛	.96	.29	.26	.44	.024	.053	.032	.057	.29	.44	.22	.....	75	40
1⅜×⅞	3/16	1.32	.39	.24	.49	.022	.071	.035	.081	.24	.43	.19	.....	110	45
	⅛	.91	.27	.22	.47	.017	.051	.026	.056	.25	.44	.20	.....	75	35
1¼×⅞	⅛	.85	.25	.23	.41	.016	.039	.024	.047	.25	.40	.19	.....	60	30
1 1/16×11/16	3/16	1.08	.32	.24	.37	.015	.033	.027	.048	.22	.32	.16	.....	64	35
1×¾	3/16	1.00	.30	.23	.35	.013	.027	.025	.042	.21	.30	.16	.....	55	30
	⅛	.70	.21	.21	.33	.0094	.020	.017	.030	.22	.31	.16	.....	40	20
1×⅝	3/16	.92	.27	.19	.38	.0074	.025	.017	.041	.17	.31	.13	.....	55	20
	⅛	.64	.19	.17	.35	.0055	.019	.012	.029	.17	.31	.13	.....	40	16
⅞×½	.095	.42	.13	.13	.31	.0022	.0093	.0054	.017	.13	.28	.12	.....	20	7
13/16×½	5/32	.62	.19	.15	.31	.0032	.011	.0091	.022	.13	.25	.11	.....	30	12

TABLE 25

Areas of Angles

Areas in Square Inches
Dimensions in Inches

Angles with Equal Legs

Size	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	Size
8″×8″	....	....	....	....	....	....	7.75	8.68	9.61	10.53	11.44	12.34	13.23	14.12	15.00	15.87	16.73	8″×8″
6 ×6	....	....	....	....	4.36	5.06	5.75	6.43	7.11	7.78	8.44	9.09	9.73	10.37	11.00	....	....	6 ×6
5 ×5	....	....	....	....	3.61	4.18	4.75	5.31	5.86	6.40	6.94	7.47	7.98	8.50	9.00	....	....	5 ×5
4 ×4	....	....	....	2.40	2.86	3.31	3.75	4.18	4.61	5.03	5.44	5.84	....	....	....	....	....	4 ×4
3½×3½	....	....	....	2.09	2.48	2.87	3.25	3.62	3.98	4.34	4.69	5.03	....	....	....	....	....	3½×3½
3 ×3	....	....	1.44	1.78	2.11	2.43	2.75	3.06	3.36	....	....	....	....	....	....	....	....	3 ×3
2¾×2¾	....	....	1.31	1.62	1.92	2.22	2.50	....	....	....	....	....	....	....	....	....	....	2¾×2¾
2½×2½	....	0.90	1.19	1.47	1.73	2.00	2.25	....	....	....	....	....	....	....	....	....	....	2½×2½
2¼×2¼	....	0.81	1.06	1.31	1.55	1.78	2.00	....	....	....	....	....	....	....	....	....	....	2¼×2¼
2 ×2	....	0.71	0.94	1.15	1.36	1.56	....	....	....	....	....	....	....	....	....	....	....	2 ×2
1¾×1¾	....	0.62	0.81	1.00	1.17	1.34	....	....	....	....	....	....	....	....	....	....	....	1¾×1¾
1½×1½	0.36	0.53	0.69	0.84	0.98	....	....	....	....	....	....	....	....	....	....	....	....	1½×1½
1¼×1¼	0.30	0.43	0.56	0.68	....	....	....	....	....	....	....	....	....	....	....	....	....	1¼×1¼
1 ×1	0.23	0.34	0.44	....	....	....	....	....	....	....	....	....	....	....	....	....	....	1 ×1

Angles with Unequal Legs

Size	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	Size
7″×3½″	....	....	....	....	....	4.40	5.00	5.59	6.17	6.75	7.31	7.87	8.42	8.97	9.50	....	....	7″×3½″
6 ×4	....	....	....	....	3.61	4.18	4.75	5.31	5.86	6.40	6.94	7.47	7.98	8.50	9.00	....	....	6 ×4
6 ×3½	....	....	....	....	3.42	3.97	4.50	5.03	5.55	6.06	6.56	7.06	7.55	8.03	8.50	....	....	6 ×3½
5 ×4	....	....	....	....	3.23	3.75	4.25	4.75	5.23	5.72	6.19	6.65	7.11	....	....	....	....	5 ×4
5 ×3½	....	....	....	2.56	3.05	3.53	4.00	4.47	4.92	5.37	5.81	6.25	6.67	....	....	....	....	5 ×3½
5 ×3	....	....	....	2.40	2.86	3.31	3.75	4.18	4.61	5.03	5.44	5.84	....	....	....	....	....	5 ×3
4 ×3½	....	....	....	2.25	2.67	3.09	3.50	3.90	4.30	4.68	5.06	5.43	....	....	....	....	....	4 ×3½
4 ×3	....	....	....	2.09	2.48	2.87	3.25	3.62	3.98	4.34	4.69	5.03	....	....	....	....	....	4 ×3
3½×3	....	....	....	1.93	2.30	2.65	3.00	3.34	3.67	4.00	4.31	4.62	....	....	....	....	....	3½×3
3½×2½	....	....	1.44	1.78	2.11	2.43	2.75	3.06	3.36	3.65	....	....	....	....	....	....	....	3½×2½
3 ×2½	....	....	1.31	1.62	1.92	2.22	2.50	2.78	....	....	....	....	....	....	....	....	....	3 ×2½
3 ×2	....	....	1.19	1.47	1.73	2.00	2.25	....	....	....	....	....	....	....	....	....	....	3 ×2
2½×2	....	0.81	1.06	1.31	1.55	1.78	2.00	....	....	....	....	....	....	....	....	....	....	2½×2
Size	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	Size

TABLE 26

Weights of Angles

Angles with Equal Legs

Weights in Pounds per Foot
Dimensions in Inches

Size	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	Size
8″×8″	....	....	....	....	....	....	26.4	29.6	32.7	35.8	38.9	42.0	45.0	48.1	51.0	54.0	56.9	8″×8″
6 ×6	....	....	....	....	14.9	17.2	19.6	21.9	24.2	26.5	28.7	31.0	33.1	35.3	37.4	....	....	6 ×6
5 ×5	....	....	....	....	12.3	14.3	16.2	18.1	20.0	21.8	23.6	25.4	27.2	28.9	30.6	....	....	5 ×5
4 ×4	....	....	....	8.2	9.8	11.3	12.8	14.3	15.7	17.1	18.5	19.9	....	....	....	....	....	4 ×4
3½×3½	....	....	....	7.2	8.5	9.8	11 1	12.4	13.6	14.8	16.0	17.1	....	....	....	....	....	3½×3½
3 ×3	....	....	4.9	6.1	7.2	8.3	9.4	10.4	11.5	....	....	....	....	....	....	....	....	3 ×3
2¾×2¾	....	....	4.5	5.6	6.6	7.6	8.5	....	....	....	....	....	....	....	....	....	....	2¾×2¾
2½×2½	....	3.1	4.1	5.0	5.9	6.8	7.7	....	....	....	....	....	....	....	....	....	....	2½×2½
2¼×2¼	....	2.8	3.6	4.5	5.3	6.1	6.8	....	....	....	....	....	....	....	....	....	....	2¼×2¼
2 ×2	....	2.4	3.2	3.9	4.7	5.3	....	....	....	....	....	....	....	....	....	....	....	2 ×2
1¾×1¾	....	2.1	2.8	3.4	4.0	4.6	....	....	....	....	....	....	....	....	....	....	....	1¾×1¾
1½×1½	1.2	1.8	3.3	2.9	3.4	....	....	....	....	....	....	....	....	....	....	....	....	1½×1½
1¼×1¼	1.0	1.5	1.9	2.3	....	....	....	....	....	....	....	....	....	....	....	....	....	1¼×1¼
1×1	0.8	1.2	1.5	....	....	....	....	....	....	....	....	....	....	....	....	....	....	1 ×1

Angles with Unequal Legs

Size	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	Size
7″×3½″	....	....	....	....	....	15.0	17.0	19.1	21.0	23.0	24.9	26.8	28.7	30.5	32.3	....	....	7″×3½″
6 ×4	....	....	....	....	12.3	14.3	16.2	18.1	20.0	21.8	23.6	25.4	27.2	28.9	30.6	....	....	6 ×4
6 ×3½	....	....	....	....	11.7	13.5	15.3	17.1	18.9	20.6	22.4	24.0	25.7	27.3	28.9	....	....	6 ×3½
5 ×4	....	....	....	....	11.0	12.8	14.5	16.2	17.8	19.5	21.1	22.7	24.2	....	....	....	....	5 ×4
5 ×3½	....	....	....	8.7	10.4	12.0	13.6	15.2	16.8	18.3	19.8	21.3	22.7	....	....	....	....	5 ×3½
5 ×3	....	....	....	8.2	9.8	11.3	12.8	14.3	15.7	17.1	18.5	19.9	....	....	....	....	....	5 ×3
4 ×3½	....	....	....	7.7	9.1	10.6	11.9	13.3	14.7	16.0	17.3	18.5	....	....	....	....	....	4 ×3½
4 ×3	....	....	....	7.2	8.5	9.8	11.1	12.4	13.6	14.8	16.0	17.1	....	....	....	....	....	4 ×3
3½×3	....	....	....	6.6	7.9	9.1	10.2	11.4	12.5	13.6	14.7	15.8	....	....	....	....	....	3½×3
3½×2½	....	....	4.9	6.1	7.2	8.3	9.4	10.4	11.5	12.5	....	....	....	....	....	....	....	3½×2½
3 ×2½	....	....	4.5	5.6	6.6	7.6	8.5	9.5	....	....	....	....	....	....	....	....	....	3 ×2½
3 ×2	....	....	4.1	5.0	5.9	6.8	7.7	....	....	....	....	....	....	....	....	....	....	3 ×2
2½×2	....	2.8	3.7	4.5	5.3	6.1	6.8	....	....	....	....	....	....	....	....	....	....	2½×2
Size	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	Size

TABLE 27

OVERRUN OF PENCOYD ANGLES

Size of Angle	Overrun of Angles in Inches																
	Thickness in Inches																
Inches	$1\frac{1}{8}$	$1\frac{1}{16}$	1	$\frac{15}{16}$	$\frac{7}{8}$	$\frac{13}{16}$	$\frac{3}{4}$	$\frac{11}{16}$	$\frac{5}{8}$	$\frac{9}{16}$	$\frac{1}{2}$	$\frac{7}{16}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$
8×8	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....	....	....	....	....
6×6	....	....	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{1}{16}$	0	0	$\frac{1}{16}$	0	0	$\frac{1}{16}$	0	....	....	....	....
4×4	....	....	....	....	....	....	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....	....
$3\frac{1}{2} \times 3\frac{1}{2}$	....	....	....	....	....	....	....	....	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{1}{8}$	$\frac{1}{16}$	0	0	....	....
3×3	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
$2\frac{1}{2} \times 2\frac{1}{2}$	....	....	....	....	....	....	....	....	....	....	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	....
2×2	....	....	....	....	....	....	....	....	....	....	....	....	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	....
$1\frac{3}{4} \times 1\frac{3}{4}$	....	....	....	....	....	....	....	....	....	....	....	....	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	....
$1\frac{1}{2} \times 1\frac{1}{2}$	....	....	....	....	....	....	....	....	....	....	....	....	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0
8×6	....	....	$\frac{1}{2}$	$\frac{7}{16}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....	....	....	....	....
$7 \times 3\frac{1}{2}$	....	....	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....	....	....	....
6×4	....	....	$\frac{7}{16}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....	....	....
$6 \times 3\frac{1}{2}$	....	....	$\frac{7}{16}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....	....	....
5×4	....	....	....	....	....	....	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....	....	....
$5 \times 3\frac{1}{2}$	....	....	....	....	....	....	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....	....
5×3	....	....	....	....	....	....	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....	....
$4 \times 3\frac{1}{2}$	....	....	....	....	....	....	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....	....
4×3	....	....	....	....	....	....	....	....	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....	....
$3\frac{1}{2} \times 3$	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
$3\frac{1}{2} \times 2\frac{1}{2}$	....	....	....	....	....	....	....	....	....	....	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	0	....	....
$3 \times 2\frac{1}{2}$	....	....	....	....	....	....	....	....	....	....	$\frac{1}{8}$	$\frac{1}{16}$	0	$\frac{1}{16}$	0	....	....
3×2	....	....	....	....	....	....	....	....	....	....	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	....	....
$2\frac{1}{2} \times 2$	....	....	....	....	....	....	....	....	....	....	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{16}$	0	....
$2 \times 1\frac{3}{8}$	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....

TABLE 28

Overrun of Pennsylvania Steel Co. Angles

Overrun of Angles in Inches																		
Size of Angle	Thickness in Inches																	Maximum Length of Angles
Inches	1 1/8	1 1/16	1	15/16	7/8	13/16	3/4	11/16	5/8	9/16	1/2	7/16	3/8	5/16	1/4	3/16	1/8	Feet
8 × 8	3/8	3/8	5/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	...	...	...	...	...	...	56 for 1 1/8″ to 105 for 1/2″
6 × 6	...	...	3/16	1/8	1/16	1/16	1/16	...	...	...	...	...	...	...	...	...	...	88 for 1″ to 105 for 5/16″
5 × 5	...	...	...	...	3/8	3/8	3/8	1/4	3/16	3/16	1/8	1/8	1/16	...	...	...	...	70
4½ × 4½	...	...	...	...	...	...	...	1/4	1/4	1/4	1/4	3/16	3/16	1/8	...	...	...	70
4 × 4	...	...	...	...	1/2	7/16	3/8	5/16	1/4	3/16	1/8	1/16	1/16	1/16	...	...	...	70
3½ × 3½	...	...	...	...	7/16	3/8	5/16	1/4	1/4	3/16	1/8	1/16	1/16	...	...	...	...	70
3 × 3	...	...	...	...	...	...	...	...	9/32	3/16	1/8	1/8	1/16	1/16	...	...	...	70
2½ × 2½	...	...	...	...	...	...	...	...	...	...	1/4	1/8	1/8	0	1/16	...	...	35 for 1/2″ to 50 for 5/16″
2¼ × 2¼	...	...	...	...	...	...	...	...	...	...	...	...	1/8	1/8	1/8	...	...	50
2 × 2	...	...	...	...	...	...	...	...	...	...	...	...	3/16	1/8	3/16	1/8	...	50
1½ × 1½	...	...	...	...	...	...	...	...	...	...	...	...	1/8	1/8	1/16	1/32	...	50
8 × 6	1/4	3/16	1/16	1/16	1/16	...	...	...	...	...	...	...	...	...	...	...	...	63 for 1 1/8″ to 105 for 1/2″
6 × 4	...	...	3/8	5/16	1/4	1/8	1/16	1/8	1/8	1/16	1/8	1/16	...	...	...	...	...	70
6 × 3½	...	...	...	...	5/16	1/4	1/4	1/4	1/4	1/4	3/16	1/8	1/16	...	...	...	...	70
5 × 4	...	...	...	...	...	...	7/16	3/8	5/16	3/8	7/16	3/8	3/16	1/8	...	...	...	70
5 × 3½	...	...	...	...	7/16	3/8	5/16	5/16	5/16	1/4	1/8	1/16	1/16	...	...	...	...	70
5 × 3	...	...	...	...	1/2	...	7/16	3/8	5/16	1/4	1/4	1/8	1/16	1/16	...	...	...	70
4½ × 3	...	...	...	...	...	...	3/8	23/64	11/32	11/32	5/16	1/4	1/4	3/16	...	...	...	70
4 × 3½	...	...	...	...	...	...	11/32	5/16	5/16	5/16	1/4	3/16	5/32	1/16	1/16	...	...	70
4 × 3	...	...	...	...	...	...	3/8	1/4	7/32	3/16	3/16	1/8	1/16	1/16	...	...	...	70
3½ × 3	...	...	...	...	...	...	...	3/8	3/8	1/4	1/4	1/8	1/16	1/16	...	...	...	70
3½ × 2½	...	...	...	...	...	...	...	...	1/2	1/4	3/8	5/16	1/8	1/16	1/32	...	...	70
3 × 2½	...	...	...	...	...	...	...	...	5/16	1/4	1/8	1/8	1/16	1/32	...	...	...	70
3 × 2	...	...	...	...	...	...	...	...	...	...	5/16	1/4	1/4	3/16	1/8	...	...	65

TABLE 29.

Carnegie Angles.

Net Areas and Allowable Tension Values in Thousands of Pounds.

Maximum Fiber Stress, 16,000 Pounds per Square Inch.

Size, Inches	Thickness, Inches.	Weight per Foot, Pounds.	Area, Inches².	Net Areas and Stresses—Two Holes Deducted.					
				⅞ Inch Rivets.		¾ Inch Rivets.		⅝ Inch Rivets.	
				Area, Inches².	Stress.	Area, Inches².	Stress.	Area, Inches².	Stress.
8 × 8	1	51.0	15.00	13.00	208.0	13.25	212.0	....	.....
8 × 8	15/16	48.1	14.12	12.24	195.8	12.48	199.7	....	.....
8 × 8	7/8	45.0	13.23	11.48	183.7	11.70	187.2	....	.....
8 × 8	13/16	42.0	12.34	10.72	171.5	10.92	174.7	....	.....
8 × 8	3/4	38.9	11.44	9.94	159.0	10.13	162.1	....	.....
8 × 8	11/16	35.8	10.53	9.16	146.6	9.33	149.3	....	.....
8 × 8	5/8	32.7	9.61	8.36	133.8	8.52	136.3	8.67	138.7
8 × 8	9/16	29.6	8.68	7 55	120.8	7.70	123.2	7.84	125.4
8 × 8	1/2	26.4	7.75	6 75	108.0	6.87	109.9	7.00	112.0
8 × 6	1	44.2	13.00	11.00	176.0	11.25	180.0	....	.....
8 × 6	15/16	41.7	12.25	10.37	165.9	10.61	169.8	....	.....
8 × 6	7/8	39.1	11.48	9 73	155.7	9.95	159.2	....	.....
8 × 6	13/16	36.5	10.72	9.10	145.6	9.30	148.8	....	.....
8 × 6	3/4	33.8	9.94	8.44	135.0	8.63	138.1	....	.....
8 × 6	11/16	31.2	9.15	7.78	124.5	7.95	127.2	....	.....
8 × 6	5/8	28.5	8.36	7.11	113.8	7.27	116.3	7.42	118.7
8 × 6	9/16	25.7	7.56	6.43	102.9	6.58	105.3	6.72	107.5
8 × 6	1/2	23.0	6.75	5.75	92.0	5.87	93.9	6.00	96.0
8 × 6	7/16	20.2	5.93	5.05	80.8	5.16	82.6	5.27	84.3
6 × 6	7/8	33.1	9.73	7.98	127.7	8.20	131.2	....	.....
6 × 6	13/16	31.0	9.09	7 47	119.5	7.67	122.7	....	.....
6 × 6	3/4	28.7	8.44	6.94	111.0	7.13	114.1	....	.....
6 × 6	11/16	26.5	7.78	6.41	102.6	6.58	105.3	....	.....
6 × 6	5/8	24.2	7.11	5.86	93.8	6.02	96.3	6.17	98.7
6 × 6	9/16	21.9	6.43	5.30	84.8	5.45	87.2	5.59	89.4
6 × 6	1/2	19.6	5.75	4.75	76.0	4.87	77.9	5.00	80.0
6 × 6	7/16	17.2	5.06	4.18	66.9	4.29	68.6	4.40	70.4
6 × 6	3/8	14.9	4.36	3 61	57.8	3.70	59.2	3.80	60.8
6 × 4	7/8	27.2	7.98	6.23	99.7	6.45	103.2	....	.....
6 × 4	13/16	25.4	7.47	5.85	93.6	6.05	96.8	....	.....
6 × 4	3/4	23.6	6.94	5.44	87.0	5.63	90.1	....	.....
6 × 4	11/16	21.8	6 40	5.03	80.5	5.20	83.2	....	.....
6 × 4	5/8	20.0	5.86	4.61	73.8	4.77	76.3	4.92	78.7
6 × 4	9/16	18.1	5.31	4.18	66.9	4.33	69.3	4.47	71.5
6 × 4	1/2	16.2	4.75	3.75	60.0	3.87	61.9	4.00	64.0
6 × 4	7/16	14.3	4.18	3.30	52.8	3.41	54.6	3.52	56.3
6 × 4	3/8	12.3	3.61	2.86	45.8	2.95	47.2	3.05	48.8
5 × 3½	5/8	16.8	4.92	3.67	58.7	3.83	61.3	3.98	63.7
5 × 3½	9/16	15.2	4.47	3.34	53.4	3.49	55.8	3.63	58.1
5 × 3½	1/2	13.6	4.00	3.00	48.0	3.12	49.9	3.25	52.0
5 × 3½	7/16	12.0	3.53	2.65	42.4	2.76	44.2	2 87	45.9
5 × 3½	3/8	10.4	3.05	2.30	36.8	2.39	38.2	2.49	39.8
5 × 3½	5/16	8.7	2.56	1.93	30.9	2.01	32.2	2.09	33.4
5 × 3	1/2	12.8	3.75	2.75	44.0	2.87	45.9	3.00	48.0
5 × 3	7/16	11.3	3.31	2.43	38.9	2.54	40.6	2.65	42.4
5 × 3	3/8	9.8	2.86	2.11	33.8	2.20	35.2	2.30	36.8
5 × 3	5/16	8.2	2.40	1.77	28.3	1 85	29.6	1.93	30.9

TABLE 29.—*Continued.*

CARNEGIE ANGLES.

NET AREAS AND ALLOWABLE TENSION VALUES IN THOUSANDS OF POUNDS.

Maximum Fiber Stress, 16,000 Pounds per Square Inch.

Size, Inches.	Thickness, Inches.	Weight per Foot, Pounds.	Area, Inches².	Net Areas and Stresses—One Hole Deducted.					
				⅞ Inch Rivets.		¾ Inch Rivets.		⅝ Inch Rivets.	
				Area, Inches².	Stress.	Area, Inches².	Stress.	Area, Inches².	Stress.
6 × 6	7/8	33.1	9.73	8.85	141.6	8.96	143.4	....	......
6 × 6	13/16	31.0	9.09	8.28	132.5	8.38	134.1	....	......
6 × 6	3/4	28.7	8.44	7.69	123.0	7.78	124.5	....	......
6 × 6	11/16	26.5	7.78	7.09	113.4	7.18	114.9	....	......
6 × 6	5/8	24.2	7.11	6.48	103.7	6.56	105.0	6.64	106.2
6 × 6	9/16	21.9	6.43	5.87	93.9	5.94	95.0	6.01	96.2
6 × 6	1/2	19.6	5.75	5.25	84.0	5.31	85.0	5.37	85.9
6 × 6	7/16	17.2	5.06	4.62	73.9	4.68	74.9	4.73	75.7
6 × 6	3/8	14.9	4.36	3.98	63.7	4.03	64.5	4.08	65.3
6 × 4	7/8	27.2	7.98	7.10	113.6	7.21	115.4	....	......
6 × 4	13/16	25.4	7.47	6.66	106.6	6.76	108.2	....	......
6 × 4	3/4	23.6	6.94	6.19	99.0	6.28	100.5	....	......
6 × 4	11/16	21.8	6.40	5.71	91.4	5.80	92.8	....	......
6 × 4	5/8	20.0	5.86	5.23	83.7	5.31	85.0	5.39	86.2
6 × 4	9/16	18.1	5.31	4.75	76.0	4.82	77.1	4.89	78.2
6 × 4	1/2	16.2	4.75	4.25	68.0	4.31	69.0	4.37	69.9
6 × 4	7/16	14.3	4.18	3.74	59.8	3.80	60.8	3.85	61.6
6 × 4	3/8	12.3	3.61	3.23	51.7	3.28	52.5	3.33	53.3
5 × 3½	5/8	16.8	4.92	4.29	68.6	4.37	69.9	4.45	71.2
5 × 3½	9/16	15.2	4.47	3.91	62.6	3.98	63.7	4.05	64.8
5 × 3½	1/2	13.6	4.00	3.50	56.0	3.56	57.0	3.62	57.9
5 × 3½	7/16	12.0	3.53	3.09	49.4	3.15	50.4	3.20	51.2
5 × 3½	3/8	10.4	3.05	2.67	42.7	2.72	43.5	2.77	44.3
5 × 3½	5/16	8.7	2.56	2.25	36.0	2.29	36.6	2.33	37.3
5 × 3	5/8	15.7	4.61	3.98	63.7	4.06	65.0	4.14	66.2
5 × 3	9/16	14.3	4.18	3.62	57.9	3.69	59.0	3.76	60.2
5 × 3	1/2	12.8	3.75	3.25	52.0	3.31	53.0	3.37	53.9
5 × 3	7/16	11.3	3.31	2.87	45.9	2.93	46.9	2.98	47.7
5 × 3	3/8	9.8	2.86	2.48	39.7	2.53	40.5	2.58	41.3
5 × 3	5/16	8.2	2.40	2.09	33.4	2.13	34.1	2.17	34.7
4 × 4	5/8	15.7	4.61	3.98	63.7	4.06	65.0	4.14	66.2
4 × 4	9/16	14.3	4.18	3.62	57.9	3.69	59.0	3.76	60.2
4 × 4	1/2	12.8	3.75	3.25	52.0	3.31	53.0	3.37	53.9
4 × 4	7/16	11.3	3.31	2.87	45.9	2.93	46.9	2.98	47.7
4 × 4	3/8	9.8	2.86	2.48	39.7	2.53	40.5	2.58	41.3
4 × 4	5/16	8.2	2.40	2.09	33.4	2.13	34.1	2.17	34.7
4 × 4	1/4	6.6	1.94	1.69	27.0	1.72	27.5	1.75	28.0
4 × 3	1/2	11.1	3.25	2.75	44.0	2.81	45.0	2.87	45.9
4 × 3	7/16	9.8	2.87	2.43	38.9	2.49	39.8	2.54	40.6
4 × 3	3/8	8.5	2.48	2.10	33.6	2.15	34.4	2.20	35.2
4 × 3	5/16	7.2	2.09	1.78	28.5	1.82	29.1	1.86	29.8
4 × 3	1/4	5.8	1.69	1.44	23.0	1.47	23.5	1.50	24.0

TABLE 29.—*Continued.*

CARNEGIE ANGLES.

NET AREAS AND ALLOWABLE TENSION VALUES IN THOUSANDS OF POUNDS.

Maximum Fiber Stress, 16,000 Pounds per Square Inch.

Size, Inches.	Thickness, Inches.	Weight per Foot, Pounds.	Area, Inches².	Net Areas and Stresses—One Hole Deducted.					
				⅞ Inch Rivets.		¾ Inch Rivets.		⅝ Inch Rivets.	
				Area, Inches².	Stress.	Area, Inches².	Stress.	Area, Inches².	Stress.
3½ × 3½	⅝	13.6	3.98	3.35	53.6	3.43	54.9	3.51	56.2
3½ × 3½	9/16	12.4	3.62	3.06	49.0	3.13	50.1	3.20	51.2
3½ × 3½	½	11.1	3 25	2.75	44.0	2.81	45.0	2.87	45.9
3½ × 3½	7/16	9.8	2.87	2.43	38.9	2.49	39.8	2.54	40.6
3½ × 3½	⅜	8.5	2.48	2.10	33.6	2.15	34.4	2.20	35.2
3½ × 3½	5/16	7.2	2.09	1.78	28.5	1.82	29.1	1.86	29.8
3½ × 3½	¼	5.8	1.69	1.44	23.0	1.47	23.5	1.50	24.0
3½ × 3	½	10.2	3.00	2.50	40.0	2.56	41.0	2.62	41.9
3½ × 3	7/16	9.1	2.65	2.21	35.4	2.27	36.3	2.32	37.1
3½ × 3	⅜	7.9	2.30	1.92	30.7	1.97	31.5	2.02	32.3
3½ × 3	5/16	6.6	1.93	1.62	25.9	1.66	26.6	1.70	27.2
3½ × 3	¼	5.4	1.56	1.31	21 0	1.34	21.4	1.37	21.9
3½ × 2½	½	9.4	2.75	2.25	36.0	2.31	37.0	2.37	37.9
3½ × 2½	7/16	8.3	2.43	1.99	31.8	2.05	32.8	2.10	33.6
3½ × 2½	⅜	7.2	2.11	1.73	27.7	1.78	28.5	1.83	29.3
3½ × 2½	5/16	6.1	1.78	1.47	23.5	1.51	24.2	1.55	24.8
3½ × 2½	¼	4.9	1.44	1.19	19.0	1.22	19.5	1.25	20.0
3 × 3	½	9.4	2.75	2.25	36.0	2.31	37.0	2.37	37.9
3 × 3	7/16	8.3	2.43	1.99	31.8	2.05	32.8	2.10	33.6
3 × 3	⅜	7.2	2.11	1.73	27.7	1.78	28.5	1.83	29.3
3 × 3	5/16	6.1	1.78	1.47	23.5	1.51	24.2	1.55	24.8
3 × 3	¼	4.9	1.44	1.19	19.0	1.22	19.5	1.25	20.0
3 × 2½	⅜	6.6	1.92	1.54	24.6	1.59	25.4	1.64	26.2
3 × 2½	5/16	5.6	1.62	1.31	21.0	1.35	21.6	1.39	22.2
3 × 2½	¼	4.5	1.31	1.06	17.0	1.09	17.4	1.12	17.9
2½ × 2½	⅜	5.9	1.73	....	....	1.40	22.4	1.45	23.2
2½ × 2½	5/16	5.0	1.47	....	....	1.20	19.2	1.24	19.8
2½ × 2½	¼	4.1	1.19	....	....	0.97	15.5	1.00	16.0
2½ × 2½	3/16	3.07	0.90	....	....	0.74	11.8	0.76	12.2
2½ × 2	⅜	5.3	1.55	....	....	1.22	19.5	1.27	20.3
2½ × 2	5/16	4.5	1.31	....	....	1.04	16.6	1.08	17.3
2½ × 2	¼	3.62	1 06	....	....	0.84	13.4	0.87	13.9
2½ × 2	3/16	2.75	0.81	....	....	0.65	10.4	0.67	10.7
2 × 2	⅜	4.7	1.36	....	....	....	....	1.08	17 3
2 × 2	5/16	3.92	1.15	....	....	....	....	0.92	14.7
2 × 2	¼	3.19	0.94	....	....	....	....	0.75	12.0
2 × 2	3/16	2.44	0.71	....	....	....	....	0.57	9.1
2 × 1½	5/16	3.39	1.00	....	....	....	....	0.77	12.3
2 × 1½	¼	2.77	0.81	....	....	....	....	0.62	9.9
2 × 1½	3/16	2.12	0.62	....	....	....	....	0.48	7.7

TABLE 30

Safe Loads, in Tons, for Equal Leg Angles

American Bridge Company Standards

Size of Angle			Length of Span in Feet											
			1	2	3	4	5	6	7	8	9	10	11	12
Equal Leg Angles	8"×8"	1 1/8"	93.493	46.747	31.164	23.373	18.699	15.582	13.356	11.687	10.388	9.349	8.499	7.791
		1/2	44.640	22.320	14.880	11.160	8.928	7.440	6.377	5.580	4.960	4.464	4.058	3.720
	6"×6"	1	45.707	22.854	15.236	11.427	9.141	7.618	6.529	5.713	5.078	4.571	4.155	3.809
		3/8	18.827	9.413	6.276	4.707	3.765	3.138	2.689	2.353	2.092	1.883	1.712	1.569
	5"×5"	1	30.933	15.467	10.311	7.733	6.187	5.156	4.419	3.867	3.437	3.093	2.812	2.578
		3/8	12.907	6.453	4.302	3.227	2.581	2.151	1.844	1.613	1.434	1.291	1.173	1.075
	4"×4"	13/16	16.053	8.027	5.351	4.013	3.211	2.676	2.293	2.007	1.784	1.605	1.459	1.338
		1/4	5.600	2.800	1.867	1.400	1.120	.933	.800	.700	.622	.560	.510	.467
	3 1/2"×3 1/2"	13/16	12.000	6.000	4.000	3.000	2.400	2.000	1.714	1.500	1.333	1.200	1.091	1.000
		5/32	2.720	1.360	.907	.680	.544	.453	.388	.340	.302	.272	.247	.227
	3"×3"	5/8	6.933	3.467	2.311	1.733	1.387	1.156	.990	.867	.770	.693	.630	.578
		1/4	1.600	.800	.533	.400	.320	.267	.229	.200	.178	.160	.145	.133
	2 3/4"×2 3/4"	1/2	4.747	2.373	1.582	1.187	.949	.791	.679	.593	.527	.475	.431	.396
		1/8	1.333	.667	.444	.333	.267	.222	.190	.167	.148	.133	.121	.111
	2 1/2"×2 1/2"	1/2	3.893	1.947	1.298	.973	.779	.649	.556	.487	.433	.389	.354	.324
		1/8	1.067	.533	.356	.267	.213	.178	.152	.133	.118	.107	.097	.089
	2 1/4"×2 1/4"	1/2	3.093	1.546	1.031	.773	.619	.515	.442	.387	.344	.309	.281	.258
		1/8	.853	.427	.284	.213	.171	.142	.122	.107	.095	.085	.078	.071
	2"×2"	7/16	2.133	1.067	.711	.533	.427	.356	.305	.267	.237	.213	.194	.178
		1/8	.693	.347	.231	.173	.139	.116	.099	.087	.077	.069	.063	.058
	1 3/4"×1 3/4"	7/16	1.600	.800	.533	.400	.320	.267	.229	.200	.178	.160	.145	.133
		1/8	.533	.267	.178	.133	.107	.089	.076	.067	.059	.053	.048	.044
	1 1/2"×1 1/2"	3/8	1.013	.507	.338	.253	.203	.169	.145	.127	.113	.101	.092	.084
		1/8	.384	.192	.128	.096	.077	.064	.055	.048	.043	.038	.035	.032
	1 1/4"×1 1/4"	5/16	.587	.293	.196	.147	.117	.098	.084	.073	.065	.059	.053	.049
		1/8	.261	.131	.087	.065	.052	.044	.037	.033	.029	.026	.024	.022
	1 1/8"×1 1/8"	3/16	.304	.152	.101	.076	.061	.051	.043	.038	.034	.030	.028	.025
		1/8	.213	.107	.071	.053	.043	.036	.030	.027	.024	.021	.019	.018
	1"×1"	1/4	.299	.149	.099	.075	.060	.050	.043	.037	.033	.029	.027	.025
		.109	.149	.075	.050	.037	.030	.025	.021	.019	.017	.015	.013	.012
	7/8"×7/8"	3/16	.176	.088	.059	.044	.035	.029	.025	.022	.020	.018	.016	.015
		3/32	.096	.048	.032	.024	.019	.016	.014	.012	.011	.010	.009	.008
	3/4"×3/4"	3/16	.128	.064	.043	.032	.026	.021	.018	.016	.014	.013	.012	.011
		3/32	.069	.035	.023	.017	.014	.012	.010	.009	.008	.007	.006	.006
	5/8"×5/8"	1/8	.060	.030	.020	.015	.012	.010	.009	.007	.007	.006	.005	.005
		3/32	.047	.023	.016	.012	.009	.008	.007	.006	.005	.005	.004	.004
	1/2"×1/2"	1/8	.037	.019	.012	.009	.007	.006	.005	.005	.004	.004	.003	.003
		3/32	.029	.015	.010	.007	.006	.005	.004	.004	.003	.003	.003	.002

Safe Load in tons of 2000 pounds uniformly distributed, for maximum fiber stress of 16,000 pounds per square inch. The Safe Load includes weight of Angle. The Safe Load for Angles of intermediate thickness can be assumed as approximately proportional to their area or weight.

TABLE 31

Safe Loads, in Tons, for Unequal Leg Angles

American Bridge Company Standards

	Size of Angle		Vertical Leg	Length of Span in Feet											
				1	2	3	4	5	6	7	8	9	10	11	12
Unequal Leg Angles	8"×6"	1"	8	80.586	40.293	26.862	20.147	16.117	13.431	11.512	10.073	8.954	8.058	7.326	6.715
			6	47.573	23.786	15.857	11.893	9.515	7.928	6.796	5.946	5.286	4.757	4.325	3.964
		7/16	8	37.706	18.853	12.568	9.426	7.541	6.284	5.387	4.713	4.189	3.771	3.428	3.142
			6	22.560	11.280	7.520	5.640	4.512	3 760	3.222	2.820	2.567	2.256	2.051	1.880
	8"×3½"	1	8	73.488	36.744	24.496	18.372	14.696	12.500	10.498	9.186	8.165	7.349	6.681	6.250
			3½	16.079	8.039	5.359	4.020	3.216	2.679	2 297	2.010	1.786	1.608	1.461	1.340
		7/16	8	34.312	17.156	11.437	8.578	6.862	5.718	4.901	4.289	3.812	3.431	3.119	2.859
			3½	7.801	3.900	2.600	1.950	1.560	1.300	1.114	0.975	0.867	0.780	0.709	0.650
	7"×3½"	1	7	56.427	28.213	18.819	14.107	11.285	9.404	8.061	7.053	6.270	5.643	5.130	4.702
			3½	15.787	7.893	5.262	3.947	3.157	2.631	2.255	1.973	1.754	1.579	1.435	1.316
		3/8	7	23.093	11.547	7.698	5.773	4.619	3.845	3.299	2.887	2.566	2.309	2.099	1.924
			3½	6.720	3.360	2.240	1.680	1.344	1.120	.960	.840	.747	.672	.611	.560
	6"×4"	1	6	42.773	21.387	14.257	10.693	8.555	7.129	6.110	5 347	4.753	4.277	3.888	3.564
			4	20.213	10.107	6.738	5.053	4.043	3.369	2.888	2.527	2.246	2 021	1.838	1.684
		3/8	6	17.707	8.853	5.902	4.427	3.541	2.951	2.529	2.213	1.967	1.771	1.609	1.476
			4	8.533	4.267	2.844	2.133	1.707	1.422	1.219	1.067	.948	.853	.776	.711
	6"×3½"	1	6	41.760	20.880	13.920	10.440	8.352	6.960	5.966	5.220	4.640	4.176	3.796	3.480
			3½	15.467	7.733	5.156	3.867	3.093	2.578	2.209	1.933	1.719	1.546	1.407	1.289
		5/16	6	14.613	7.307	4.871	3.653	2.923	2.435	2.087	1.827	1.624	1.461	1.328	1.218
			3½	5.547	2.773	1.848	1.386	1.109	.924	.792	.693	.616	.555	.504	.462
	5"×4"	7/8	5	26.613	13.306	8.871	6.653	5.323	4.435	3.802	3.327	2.957	2.661	2.418	2.217
			4	17.653	8.826	5.884	4.413	3.531	2.942	2.522	2.207	1.961	1.765	1.605	1.471
		3/8	5	12.480	6.240	4.160	3.120	2.496	2.080	1.783	1.560	1.387	1.248	1.134	1.040
			4	8.373	4.186	2.791	2.093	1.675	1.395	1.196	1.046	.930	.837	.761	.697
	5"×3½"	7/8	5	26.026	13.013	8.675	6.506	5.205	4.338	3.718	3.253	2.892	2.603	2.366	2.160
			3½	13.440	6.720	4.480	3.360	2.688	2.240	1.920	1.680	1.493	1.344	1.222	1.120
		5/16	5	10.346	5.173	3.449	2.587	2.069	1.724	1.478	1.293	1.149	1.035	.941	.844
			3½	5.440	2.720	1.813	1.360	1.088	.907	.777	.680	.604	.544	.494	.459
	5"×3"	13/16	5	23.733	11.867	7.911	5.933	4.747	3.955	3.390	2.967	2.637	2.373	2.157	1.977
			3	9.280	4.640	3.093	2.320	1.856	1.546	1.326	1.160	1.031	.928	.843	.773
		5/16	5	10.080	5.040	3.360	2.520	2.016	1.680	1.440	1.260	1.120	1.008	.931	.840
			3	4.000	2.000	1.333	1.000	.800	.666	.571	.500	.444	.400	.363	.333
	4½"×3"	13/16	4½	19.306	9.653	6.433	4.827	3.861	3.217	2.758	2.413	2.145	1.931	1.755	1.689
			3	9.120	4.560	3.040	2.280	1.824	1.520	1.303	1.140	1.013	.912	.829	.760
		5/16	4½	8.213	4.106	2.738	2.053	1.643	1.369	1.173	1.027	.913	.821	.747	.684
			3	4.000	2.000	1.333	1.000	.800	.666	.571	.500	.444	.400	.363	.333

Safe Load in Tons of 2000 pounds uniformly distributed, for maximum fiber stress of 16,000 pounds per square inch. The Safe Load includes weight of Angle. The Safe Load for Angles of intermediate thickness can be assumed as proportional to their area or weight.

TABLE 31.—*Continued*

Safe Loads, in Tons, for Unequal Leg Angles
American Bridge Company Standards

	Size of Angle		Vertical Leg	Length of Span in Feet											
				1	2	3	4	5	6	7	8	9	10	11	12
Unequal Leg Angles	4″×3½″	13/16″	4	15.573	7.787	5.191	3.893	3.115	2.595	2.225	1.947	1.730	1.558	1.416	1.298
			3½	12.267	6.133	4.089	3.067	2.453	2.044	1.752	1.533	1.363	1.227	1.115	1.022
		5/16	4	6.720	3.360	2.240	1.680	1.344	1.120	.960	.840	.747	.672	.619	.560
			3½	5.333	2.667	1.778	1.333	1.067	.889	.768	.667	.592	.533	.485	.444
	4″×3″	13/16	4	15.307	7.653	5.102	3.827	3.061	2.551	2.187	1.913	1.701	1.531	1.391	1.275
			3	8.960	4.480	2.987	2.240	1.792	1.493	1.280	1.120	.995	.896	.814	.747
		1/4	4	5.333	2.667	1.778	1.333	1.067	.889	.762	.667	.593	.533	.485	.444
			3	3.200	1.600	1.067	.800	.640	.533	.457	.400	.355	.320	.297	.267
	4″×2½″	5/8	4	11.627	5.813	3.875	2.907	2.325	1.938	1.661	1.453	1.291	1.163	1.057	.969
			2½	4.053	2.026	1.351	1.013	.811	.675	.599	.507	.451	.405	.368	.338
		3/8	4	7.413	3.707	2.471	1.853	1.483	1.235	1.059	.927	.824	.741	.674	.618
			2½	2.613	1.307	.871	.653	.523	.435	.373	.327	.290	.261	.237	.218
	4″×2″	3/8	4	7.253	3.627	2.418	1.813	1.451	1.209	1.036	.907	.806	.725	.659	.604
			2	2.027	1.013	.675	.507	.405	.338	.289	.253	.225	.203	.184	.169
		1/4	4	5.013	2.507	1.671	1.253	1.003	.835	.716	.627	.557	.501	.456	.418
			2	1.440	.720	.480	.360	.288	.240	.206	.180	.160	.144	.131	.120
	3½″×3″	13/16	3½	11.733	5.867	3.911	2.933	2.347	1.955	1.676	1.467	1.304	1.173	1.067	.978
			3	8.800	4.400	2.933	2.200	1.760	1.578	1.257	1.100	.978	.880	.800	.733
		1/4	3½	4.160	2.080	1.387	1.040	.832	.693	.594	.520	.462	.416	.378	.344
			3	3.093	1.547	1.031	.773	.619	.515	.442	.387	.344	.309	.281	.258
	3½″×2½″	11/16	3½	9.867	4.933	3.289	2.467	1.973	1.644	1.409	1.233	1.096	.987	.897	.822
			2½	5.280	2.640	1.760	1.320	1.056	.880	.754	.660	.587	.528	.480	.440
		1/4	3½	4.000	2.000	1.333	1.000	.800	.666	.571	.500	.444	.400	.364	.333
			2½	2.187	1.093	.729	.547	.437	.364	.312	.273	.243	.219	.199	.182
	3½″×2″	3/8	3½	5.600	2.800	1.867	1.400	1.120	.933	.800	.700	.622	.560	.509	.467
			2	2.027	1.013	.675	.507	.405	.338	.289	.253	.225	.203	.184	.169
		1/4	3½	3.840	1.920	1.280	.960	.768	.640	.548	.480	.427	.384	.349	.320
			2	1.387	.693	.462	.347	.277	.231	.198	.173	.154	.149	.126	.115
	3¼″×2″	9/16	3¼	6.933	3.466	2.311	1.733	1.386	1.155	.990	.867	.770	.693	.630	.578
			2	2.827	1.413	.942	.707	.565	.471	.404	.353	.314	.283	.257	.235
		1/4	3¼	3.360	1.680	1.120	.840	.672	.560	.480	.420	.373	.336	.305	.280
			2	1.387	.693	.462	.346	.277	.231	.198	.173	.154	.148	.126	.115
	3¼″×1⅝″	3/16	3¼	2.507	1.253	.835	.627	.501	.418	.358	.313	.278	.251	.228	.209
			1⅝	.693	.347	.231	.173	.139	.115	.099	.087	.077	.069	.063	.057
	3″×2 13/16″	9/16	3	6.240	3.120	2.080	1.560	1.248	1.040	.891	.780	.693	.624	.567	.520
			2 13/16	5.547	2.773	1.849	1.387	1.109	.924	.792	.693	.616	.555	.504	.462
	3″×2 11/16″	9/16	3	6.240	3.120	2.080	1.560	1.248	1.040	.891	.780	.693	.624	.567	.520
			2 11/16	5.067	2.533	1.689	1.267	1.013	.844	.724	.633	.563	.507	.461	.422
	3″×2½″	9/16	3	6.133	3.067	2.044	1.533	1.227	1.022	.876	.767	.681	.613	.557	.511
			2½	4.373	2.187	1.458	1.093	.875	.729	.625	.547	.486	.437	.397	.364
		3/16	3	2.293	1.147	.764	.573	.459	.382	.328	.287	.255	.229	.208	.191
			2½	1.653	.827	.551	.413	.331	.275	.236	.207	.184	.165	.150	.138

Safe Load in tons of 2000 pounds uniformly distributed, for maximum fiber stress of 16,000 pounds per square inch. The Safe Load includes weight of Angle. The Safe Load for Angles of intermediate thickness can be assumed as approximately proportional to their area or weight.

TABLE 31.—*Continued*

SAFE LOADS, IN TONS, FOR UNEQUAL LEG ANGLES
AMERICAN BRIDGE COMPANY STANDARDS

Size of Angles		Vertical Leg	Length of Span in Feet: 1	2	3	4	5	6	7	8	9	10	11	12
Unequal Leg Angles														
3″×2″	1/2″	3	5.333	2.667	1.778	1.333	1.067	.889	.762	.667	.592	.533	.485	.444
		2	2.507	1.253	.835	.627	.501	.418	.358	.313	.278	.251	.228	.209
	3/16	3	2.187	1.093	.729	.547	.437	.365	.312	.273	.243	.219	.199	.182
		2	1.067	.533	.355	.267	.213	.178	.152	.133	.118	.107	.097	.089
2½″×2″	1/2	2½	3.733	1.867	1.244	.933	.747	.622	.533	.467	.415	.373	.339	.311
		2	2.453	1.227	.818	.613	.491	.409	.350	.307	.272	.245	.223	.204
	3/16	2½	1.547	.773	.515	.387	.309	.258	.221	.193	.172	.155	.141	.129
		2	1.067	.533	.355	.267	.213	.178	.152	.133	.118	.107	.097	.089
2½″×1¾″	5/16	2½	2.453	1.223	.818	.613	.491	.409	.350	.307	.272	.245	.223	.204
		1¾	1.280	.640	.427	.320	.256	.213	.183	.160	.142	.128	.116	.107
	3/16	2½	1.547	.773	.515	.387	.309	.258	.221	.193	.172	.155	.141	.129
		1¾	.800	.400	.267	.200	.160	.133	.114	.100	.089	.080	.073	.067
2½″×1½″	5/16	2½	2.347	1.173	.782	.587	.469	.391	.335	.293	.261	.235	.213	.195
		1½	.907	.453	.302	.227	.181	.151	.129	.113	.101	.091	.082	.075
	3/16	2½	1.493	.747	.497	.373	.299	.249	.213	.187	.166	.149	.136	.124
		1½	.587	.293	.195	.147	.117	.098	.084	.073	.065	.059	.053	.049
2½″×1¼″	5/32	2½	1.227	.613	.409	.307	.245	.204	.175	.153	.136	.123	.111	.102
		1¼	.352	.176	.117	.088	.070	.059	.050	.044	.039	.035	.032	.029
2¼″×1½″	1/2	2¼	2.880	1.440	.960	.720	.576	.480	.411	.360	.320	.288	.262	.240
		1½	1.387	.693	.462	.347	.277	.231	.198	.173	.154	.139	.126	.115
	3/16	2¼	1.227	.613	.409	.307	.245	.204	.175	.153	.136	.123	.111	.102
		1½	.587	.293	.195	.147	.117	.098	.084	.078	.065	.059	.053	.049
2″×1½″	3/8	2	1.813	.907	.604	.453	.363	.302	.259	.227	.201	.181	.165	.151
		1½	1.067	.533	.355	.267	.213	.178	.152	.133	.118	.107	.097	.089
	1/8	2	.693	.347	.231	.173	.139	.115	.099	.087	.077	.069	.063	.058
		1½	.400	.200	.133	.100	.080	.067	.057	.050	.044	.040	.036	.033
2″×1⅜″	3/8	2	1.760	.880	.587	.440	.352	.293	.251	.220	.195	.176	.160	.147
		1⅜	.907	.453	.302	.227	.181	.151	.129	.113	.101	.091	.082	.075
	3/16	2	.960	.480	.320	.240	.192	.160	.137	.120	.107	.096	.087	.080
		1⅜	.501	.251	.167	.125	.100	.083	.072	.063	.056	.050	.045	.042
2″×1¼″	1/4	2	1.227	.613	.409	.307	.245	.204	.175	.153	.136	.123	.111	.102
		1¼	.517	.259	.172	.129	.103	.086	.074	.065	.057	.052	.047	.043
	3/16	2	.960	.480	.320	.240	.192	.160	.137	.120	.107	.096	.087	.080
		1¼	.400	.200	.133	.100	.080	.067	.057	.050	.044	.040	.036	.033

Safe Load in Tons of 2,000 pounds uniformly distributed, for maximum fiber stress of 16,000 pounds per square inch. The Safe Load includes weight of Angle. The Safe Load for Angles of intermediate thickness can be assumed as proportional to their area or weight.

TABLE 31.—*Continued*

Safe Loads, in Tons, for Unequal Leg Angles

American Bridge Company Standards

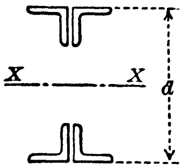
	Size of Angle		Vertical Leg	Length of Span in Feet											
				1	2	3	4	5	6	7	8	9	10	11	12
Unequal Leg Angles	$1\frac{3}{4}'' \times 1\frac{1}{4}''$	$\frac{1}{4}''$	$1\frac{3}{4}$	.960	.480	.320	.240	.192	.160	.137	.120	.107	.096	.087	.080
			$1\frac{1}{4}$	.507	.253	.169	.127	.101	.084	.072	.063	.056	.051	.046	.042
		$\frac{1}{8}$	$1\frac{3}{4}$	.501	.251	.167	.125	.100	.083	.072	.063	.056	.050	.045	.042
			$1\frac{1}{4}$	.277	.139	.092	.069	.055	.046	.040	.035	.031	.028	.025	.023
	$1\frac{3}{4}'' \times 1\frac{1}{8}''$	$\frac{1}{4}$	$1\frac{3}{4}$	.907	.453	.302	.227	.181	.151	.129	.113	.101	.091	.082	.075
			$1\frac{1}{8}$	.411	.205	.137	.103	.082	.068	.059	.051	.046	.041	.037	.034
		$\frac{1}{8}$	$1\frac{3}{4}$	.496	.248	.165	.124	.099	.083	.071	.062	.055	.050	.045	.041
			$1\frac{1}{8}$	.229	.115	.076	.057	.046	.038	.033	.029	.025	.023	.021	.019
	$1\frac{1}{2}'' \times 1\frac{1}{4}''$	$\frac{5}{16}$	$1\frac{1}{2}$	.853	.426	.284	.213	.171	.142	.122	.107	.095	.085	.077	.071
			$1\frac{1}{4}$	.603	.301	.201	.151	.120	.100	.086	.075	.067	.060	.055	.050
		$\frac{3}{16}$	$1\frac{1}{2}$	.533	.267	.178	.133	.107	.089	.076	.067	.059	.053	.048	.044
			$1\frac{1}{4}$	.389	.195	.129	.097	.078	.065	.056	.049	.043	.039	.035	.032
	$1\frac{3}{8}'' \times 1''$	$\frac{1}{4}$	$1\frac{3}{8}$	.565	.283	.188	.141	.113	.094	.081	.071	.063	.056	.051	.047
			1	.315	.157	.105	.079	.063	.052	.045	.039	.035	.031	.029	.026
		$\frac{1}{8}$	$1\frac{3}{8}$	.304	.152	.101	.076	.061	.051	.044	.038	.034	.030	.028	.025
			1	.171	.085	.057	.043	.034	.028	.024	.021	.019	.017	.015	.014
	$1\frac{3}{8}'' \times \frac{7}{8}''$	$\frac{3}{16}$	$1\frac{3}{8}$	.432	.216	.144	.108	.086	.072	.062	.054	.048	.043	.039	.036
			$\frac{7}{8}$	.187	.093	.062	.047	.037	.031	.027	.023	.021	.019	.017	.015
		$\frac{1}{8}$	$1\frac{3}{8}$	.299	.149	.099	.075	.060	.050	.043	.037	.033	.030	.027	.025
			$\frac{7}{8}$	.139	.069	.046	.035	.028	.023	.020	.017	.015	.014	.013	.011
	$1\frac{1}{4}'' \times \frac{7}{8}''$	$\frac{1}{8}$	$1\frac{1}{4}$	.251	.125	.083	.063	.050	.042	.036	.031	.028	.025	.023	.021
			$\frac{7}{8}$	.128	.064	.043	.032	.026	.021	.018	.016	.014	.013	.012	.011
	$1\frac{1}{16}'' \times \frac{13}{16}''$	$\frac{3}{16}$	$1\frac{1}{16}$	.256	.128	.085	.064	.051	.043	.036	.032	.028	.026	.023	.021
			$\frac{13}{16}$	.114	.072	.048	.036	.029	.024	.020	.018	.016	.014	.013	.012
	$1'' \times \frac{3}{4}''$	$\frac{3}{16}$	1	.224	.112	.075	.056	.045	.037	.032	.028	.025	.022	.020	.019
			$\frac{3}{4}$	.133	.067	.044	033	.027	.022	.019	.017	.015	.013	.012	.011
		$\frac{1}{8}$	1	.160	.080	.053	.040	.032	.027	.023	.020	.018	.016	.014	.013
			$\frac{3}{4}$	.091	.045	.030	.023	.018	.015	.013	.011	.010	.009	.008	.007
	$1'' \times \frac{5}{8}''$	$\frac{3}{16}$	1	.219	.109	.073	.055	.044	.036	.031	.027	.024	.022	.020	.018
			$\frac{5}{8}$	.091	.045	.030	.023	.018	.015	.013	.011	.010	.009	.008	.007
		$\frac{1}{8}$	1	.155	.077	.051	.039	.031	.026	.022	.019	.017	.015	.014	.013
			$\frac{5}{8}$	.064	.032	.021	.016	.013	.011	.009	.008	.007	.006	.006	.005
	$\frac{7}{8}'' \times \frac{1}{2}''$	.095	$\frac{7}{8}$	.091	.045	.030	.023	.018	.015	.013	.011	.010	.009	.008	.007
			$\frac{1}{2}$	.029	.014	.010	.007	.006	.005	.004	.004	.003	.003	.003	.002
	$\frac{13}{16}'' \times \frac{1}{2}''$	$\frac{5}{32}$	$\frac{13}{16}$	.117	.059	.039	.029	.023	.019	.017	.015	013	.012	.011	.010
			$\frac{1}{2}$	.048	.024	.016	.012	.010	.008	.007	.006	.005	.005	.004	.004

Safe Load in tons of 2,000 pounds uniformly distributed, for maximum fiber stress of 16,000 pounds per square inch. The Safe Load includes weight of Angle. The Safe Load for Angles of intermediate thickness can be assumed as approximately proportional to their area or weight.

TABLE 32.

Moments of Inertia of Four Angles with Equal Legs, Axis X–X.

Moments of Inertia of Four Angles, Axis X–X, Equal Legs.



For Distances Measured from Back to Back.

2½″ × 2½″						
Thick.	$\frac{3}{16}$″	¼″	$\frac{5}{16}$″	⅜″	$\frac{7}{16}$″	½″
Area 4∟s	3.60	4.76	5.88	6.92	8.00	9.00
d″	Moments of Inertia About Axis X–X, In.⁴					
5½	17	22	27	31	35	39
5¾	19	25	30	35	39	43
6	21	28	33	39	44	48
6¼	24	30	37	43	48	53
6½	26	33	40	47	53	58
6¾	28	36	44	51	58	64
7	31	40	48	56	64	70
7¼	33	43	52	61	69	76
7½	36	46	57	66	75	83
7¾	39	50	61	71	81	89
8	42	54	66	77	87	96
8¼	45	58	71	82	94	104
8½	48	62	76	88	101	111
8¾	51	66	81	94	108	119
9	54	71	87	101	115	127
9¼	58	75	92	107	123	136
9½	62	80	98	114	131	145
9¾	65	85	104	121	139	154
10	69	90	110	128	147	163
10¼	73	95	116	136	155	172
10½	77	100	123	143	164	182
10¾	81	106	130	151	173	192
11	85	112	137	159	183	203
11¼	90	117	144	168	192	214
11½	94	123	151	176	202	225
11¾	99	129	158	185	212	236
12	104	135	166	194	222	247
12¼	109	142	174	203	233	259
12½	113	148	182	216	244	271
12¾	119	155	190	222	255	283
13	124	162	198	232	266	296
13¼	129	169	207	242	278	309
13½	134	176	216	252	290	322
13¾	140	183	225	263	302	336
14	146	191	234	273	314	350
14¼	151	198	243	284	327	364
14½	157	206	253	295	339	378
14¾	163	214	262	307	352	393
15	169	222	272	318	366	408
15¼	175	230	282	330	379	423
15½	182	238	292	342	393	438
15¾	188	246	303	354	407	454

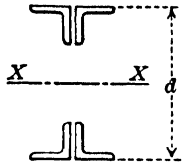
3″ × 3″							
Thick.	¼″	$\frac{5}{16}$″	⅜″	$\frac{7}{16}$″	½″	$\frac{9}{16}$″	⅝″
Area 4∟s	5.76	7.12	8.44	9.72	11.00	12.24	13.44
d″	Moments of Inertia About Axis X–X, In.⁴						
6½	38	46	54	61	68	75	80
6¾	42	50	58	67	75	83	88
7	46	55	65	73	82	89	96
7¼	50	60	70	80	89	97	104
7½	54	65	76	86	96	106	114
7¾	58	70	82	93	104	114	123
8	62	76	89	101	113	124	133
8¼	67	81	95	108	121	133	143
8½	72	87	102	116	130	143	154
8¾	77	94	110	125	139	153	165
9	82	100	117	133	149	164	177
9¼	87	106	125	142	159	175	189
9½	93	113	134	151	169	186	201
9¾	99	120	141	161	180	198	214
10	105	127	150	171	191	211	228
10¼	111	135	158	181	202	223	241
10½	117	143	167	191	214	236	256
10¾	123	151	177	202	226	249	270
11	130	159	186	213	239	263	285
11¼	137	167	196	224	251	277	300
11½	144	176	206	237	264	292	316
11¾	151	184	217	248	278	307	333
12	158	193	227	260	292	322	349
12¼	166	203	238	272	306	338	366
12½	174	212	250	285	320	354	384
12¾	181	222	261	298	335	370	402
13	189	232	273	312	350	387	420
13¼	198	242	285	325	366	404	439
13½	206	252	297	339	382	422	458
13¾	215	263	309	354	398	439	478
14	224	274	322	368	414	458	498
14¼	233	285	335	383	431	476	518
14½	242	296	348	399	448	496	539
14¾	251	307	362	414	466	515	560
15	261	319	376	430	484	535	582
15¼	270	331	390	446	502	555	604
15½	280	343	404	463	521	576	626
15¾	290	355	419	480	539	597	649
16	300	368	434	496	559	618	673
16¼	311	381	449	514	578	640	697
16½	321	394	464	532	598	662	721
16¾	332	407	480	550	619	685	745

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 32.—*Continued.*

Moments of Inertia of Four Angles with Equal Legs, Axis X–X.

Moments of Inertia of Four Angles, Axis X-X, Equal Legs.



For Distances Measured from Back to Back.

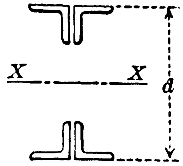
3½″×3½″

Thick.	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	Thick.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″
Area 4 ∟s	8.36	9.92	11.4[illegible]	13.00	14.48	15.92	17.36	Area 4 ∟s	9.92	11.48	13.00	14.48	15.92	17.36	18.76
d″	Moments of Inertia about Axis X-X, In.⁴							d″	Moments of Inertia about Axis X-X, In.⁴						
7½	73	86	97	109	119	129	139	20¼	836	961	1083	1201	1314	1426	1531
7¾	79	93	105	118	129	140	150	20½	858	987	1112	1234	1350	1466	1573
8	86	100	114	127	139	151	163	22¼	1026	1181	1332	1477	1617	1756	1886
8¼	92	108	122	137	150	163	175	22½	1052	1210	1364	1514	1657	1800	1934
8½	99	116	131	147	161	175	189	24¼	1237	1424	1606	1782	1952	2121	2279
8¾	106	124	141	157	173	188	203	24½	1265	1456	1642	1823	1997	2169	2331
9	113	132	150	168	185	201	217	26¼	1467	1690	1907	2117	2319	2521	2710
9¼	120	141	161	180	198	215	232	26½	1498	1725	1946	2161	2367	2573	2766
9½	128	150	171	192	211	229	247	28¼	1718	1979	2234	2480	2718	2955	3178
9¾	136	160	182	204	224	244	263	28½	1750	2016	2276	2528	2770	3011	3239
10	144	169	193	216	238	259	280	30¼	1988	2291	2586	2872	3149	3424	3684
10¼	153	179	205	229	253	275	297	30½	2023	2331	2632	2923	3205	3485	3750
10½	162	190	217	243	267	291	315	32¼	2278	2625	2965	3294	3611	3927	4227
10¾	171	200	229	257	283	308	333	32½	2315	2669	3014	3348	3671	3993	4297
11	180	211	241	271	299	325	352	34¼	2588	2983	3370	3744	4106	4466	4807
11¼	189	223	254	285	315	343	371	34½	2628	3030	3422	3802	4170	4535	4883
11½	199	234	268	301	332	362	391	36¼	2917	3364	3800	4223	4632	5039	5426
11¾	209	246	281	316	349	380	411	36½	2960	3413	3856	4285	4700	5113	5505
12	220	258	295	332	366	400	432	38¼	3267	3768	4257	4731	5190	5646	6081
12¼	230	271	310	348	385	419	453	38½	3312	3820	4316	4797	5262	5725	6166
12½	241	284	325	365	403	440	475	40¼	3636	4194	4740	5268	5780	6289	6774
12¾	252	297	340	382	422	460	498	40½	3684	4249	4802	5337	5856	6372	6864
13	264	310	355	399	441	482	521	42¼	4025	4644	5248	5834	6401	6966	7505
13¼	275	324	371	417	461	503	545	42½	4075	4702	5314	5907	6481	7053	7599
13½	287	338	387	435	481	525	569	44¼	4434	5117	5783	6429	7055	7678	8273
13¾	299	353	404	454	502	548	594	44½	4487	5177	5852	6505	7139	7769	8372
14	312	368	421	473	523	571	619	46¼	4863	5612	6344	7053	7740	8425	9079
14¼	324	383	438	493	545	595	645	46½	4918	5776	6416	7133	7828	8520	9182
14½	337	398	456	513	567	619	671	48¼	5312	6131	6930	7706	8457	9206	9922
14¾	351	414	474	533	590	644	698	48½	5369	6197	7006	7790	8549	9306	10030
15	364	430	492	554	613	669	725	50¼	5780	6672	7543	8388	9206	10022	10803
15¼	378	446	511	575	636	695	753	50½	5840	6742	7622	8475	9302	10127	10916
15½	392	462	530	596	660	721	782	52¼	6269	7237	8182	9099	9987	10873	11721
15¾	406	479	549	618	685	748	811	52½	6331	7309	8264	9189	10087	10982	11839
16	421	496	569	641	709	775	840	54¼	6777	7824	8847	9838	10800	117[illegible]8	12677
16¼	435	514	589	663	735	803	870	54½	6842	7899	8931	9933	10904	11872	12799
16½	450	532	609	687	760	831	901	56¼	7305	8435	9537	10607	11644	12679	13671
16¾	466	550	631	710	787	860	932	56½	7372	8513	9625	10705	11752	12796	13798
18	546	645	740	834	924	1011	1097	58¼	7853	9068	10254	11405	12521	13634	14701
18¼	563	665	763	860	953	1043	1131	58½	7923	9149	10345	11507	12633	13756	14833
18½	580	685	787	887	982	1075	1166	60¼	8421	9724	10997	12232	13429	14623	15770
18¾	598	706	811	913	1012	1107	1202	60½	8494	9808	11091	12338	13546	14751	15906

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

MOMENTS OF INERTIA OF FOUR ANGLES WITH EQUAL LEGS, AXIS X–X.

Moments of Inertia of Four Angles, Axis X–X, Equal Legs.



For Distances Measured from Back to Back.

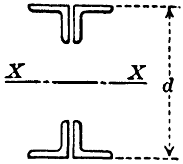
Size.	4″ × 4″													
Thick.	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	Thick.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″
Area 4∟s	9.60	11.44	13.24	15.00	16.72	18.44	Area 4∟s	11.44	13.24	15.00	16.72	18.44	20.12	21.76
d″	Moments of Inertia About Axis X–X, In.⁴.						d″	Moments of Inertia About Axis X–X, In.⁴.						
8½	109	128	146	164	179	195	24¼	1398	1612	1819	2016	2215	2408	2595
8¾	117	137	157	176	192	209	24½	1430	1648	1860	2062	2267	2463	2656
9	125	146	163	188	205	224	26¼	1661	1915	2162	2398	2636	2866	3089
9¼	133	156	179	200	219	239	26½	1695	1955	2208	2448	2692	2926	3155
9½	141	166	191	213	234	255	28¼	1946	2245	2536	2813	3093	3364	3627
9¾	150	177	203	227	249	272	28½	1984	2289	2585	2868	3154	3429	3697
10	159	188	215	241	265	289	30¼	2255	2602	2939	3262	3587	3902	4208
10¼	169	199	228	256	281	306	30½	2295	2648	2992	3320	3652	3972	4283
10½	179	211	241	271	297	325	32¼	2586	2985	3373	3744	4118	4481	4832
10¾	189	223	255	286	315	343	32½	2629	3035	3429	3807	4188	4556	4914
11	199	235	269	302	332	363	34¼	2941	3395	3836	4259	4686	5099	5501
11¼	210	248	284	319	350	383	34½	2986	3448	3896	4326	4760	5179	5587
11½	221	261	299	336	369	403	36¼	3318	3831	4329	4808	5290	5758	6212
11¾	232	274	314	353	388	424	36½	3367	3887	4393	4879	5369	5843	6304
12	243	288	330	371	408	446	38¼	3718	4293	4853	5391	5932	6457	6968
12¼	255	302	346	389	428	468	38½	3769	4353	4920	5466	6016	6548	7065
12½	267	316	363	408	449	491	40¼	4141	4782	5406	6007	6610	7197	7767
12¾	280	331	380	427	471	515	40½	4195	4845	5477	6086	6699	7292	7869
13	293	346	397	447	492	539	42¼	4587	5297	5989	6656	7325	7976	8609
13¼	306	362	415	467	515	563	42½	4644	5364	6064	6739	7418	8077	8717
13½	319	377	434	488	538	588	44¼	5[illegible]55	5839	6603	7338	8078	8796	9495
13¾	333	394	452	509	561	614	44½	5115	5909	6681	7426	8175	8902	9609
14	347	410	471	530	585	641	46¼	5547	6408	7246	8055	8867	9656	10424
14¼	361	427	491	552	609	667	46½	5610	6481	7329	8146	8969	9767	10543
14½	376	444	511	575	634	695	48¼	6061	7003	7919	8804	9693	10557	11397
14¾	390	462	531	598	660	723	48½	6127	7079	8006	8900	9799	10672	11522
15	406	480	552	621	686	752	50¼	6599	7624	8623	9587	10555	11497	12413
15¼	421	499	573	645	713	781	50½	6667	7703	8713	9687	10667	11618	12544
15½	437	517	595	670	740	810	52¼	7159	8272	9356	10404	11455	12478	13473
15¾	453	536	617	695	767	841	52½	7231	8355	9450	10508	11571	12604	13609
16	469	556	639	720	795	872	54¼	7742	8946	10119	11253	12392	13499	14577
16¼	486	576	662	746	824	903	54½	7816	9032	10217	11362	12512	13630	14718
16½	503	596	685	772	853	935	56¼	8348	9647	10913	12137	13365	14561	15724
16¾	520	616	709	799	883	968	56½	8425	9736	11014	12250	13490	14696	15870
18	611	724	834	939	1039	1141	58¼	8977	10374	11736	13054	14375	15662	16914
18¼	630	747	8[illegible]0	9[illegible]9	10[illegible]2	1176	58½	9057	10467	11841	13170	14505	15803	17066
18½	649	770	886	999	1105	1213	60¼	9629	11128	12589	14004	15423	16804	18148
18¾	669	793	913	1030	1138	1250	60½	9712	11224	12698	14125	15557	16950	18306
20¼	793	941	1084	1222	1353	1486	62¼	10303	11908	13473	14987	16507	17986	19426
20½	825	967	1114	1256	1391	1527	62½	10389	12007	13585	15113	16646	18137	19589
22¼	976	1158	1335	1506	1668	1832	64¼	11001	12715	14386	16004	17628	19208	20747
22½	1010	1187	1369	1543	1710	1879	64½	11089	12817	14502	16134	17771	19364	20915

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 32.—*Continued.*

Moments of Inertia of Four Angles with Equal Legs, Axis X-X.

Moments of Inertia of Four Angles Axis X-X, Equal Legs



For Distances Measured from Back to Back.

Size: 5″ × 5″

Thick.	3/8″	7/16″	1/2″	9/16″	5/8″
Area 4 ∟s	14.44	16.72	19.00	21.24	23.44
d″	Moments of Inertia About Axis X-X, In.⁴.				
10½	250	287	322	355	387
10¾	264	303	341	375	410
11	279	320	360	396	433
11¼	294	337	379	418	457
11½	309	355	400	441	482
11¾	325	373	420	464	507
12	342	392	442	488	533
12¼	359	412	464	512	560
12½	376	432	486	537	588
12¾	394	452	510	563	616
13	412	473	533	589	645
13¼	431	495	558	616	675
13½	450	517	583	644	705
13¾	469	540	608	673	737
14	489	563	634	702	769
14¼	510	586	661	731	801
14½	531	610	689	762	835
14¾	552	635	717	793	869
15	574	660	745	825	904
15¼	596	686	774	857	939
15½	619	712	804	890	976
15¾	642	739	834	924	1013
16	666	766	865	958	1051
16¼	690	794	897	993	1089
16½	715	822	929	1029	1129
16¾	739	851	961	1065	1169
18	871	1003	1134	1257	1380
18¼	899	1035	1170	1298	1424
18½	927	1068	1207	1339	1469
18¾	956	1101	1244	1380	1515
20¼	1137	1310	1481	1645	1806
20½	1169	1347	1523	1691	1857
22¼	1403	1618	1831	2034	2235
22½	1439	1659	1877	2085	2292
24¼	1699	1960	2218	2466	2710
24½	1738	2005	2269	2523	2773
26¼	2023	2335	2644	2940	3233
26½	2066	2384	2700	3002	3302

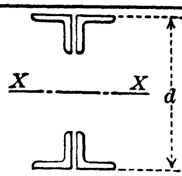
Thick.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″
Area 4 ∟s	14.44	16.72	19.00	21.24	23.44	25.60	27.76
d″	Moments of Inertia About Axis X-X, In.⁴.						
28¼	2377	2743	3107	3457	3802	4139	4474
28½	2423	2797	3168	3524	3877	4220	4562
30¼	2759	3185	3608	4016	4419	4811	5201
30½	2809	3243	3674	4089	4499	4899	5296
32¼	3170	3660	4148	4618	5082	5434	5984
32½	3224	3722	4218	4696	5168	5628	6086
34¼	3610	4169	4725	5262	5792	6309	6823
34½	3667	4235	4800	5345	5884	6409	6932
36¼	4079	4712	5341	5949	6549	7134	7717
36½	4140	4782	5420	6037	6646	7241	7833
38¼	4577	5287	5994	6678	7352	8011	8667
38½	4641	5361	6078	6772	7456	8124	8789
40¼	5103	5896	6686	7449	8203	8939	9672
40½	5171	5975	6775	7549	8313	9059	9802
42¼	5659	6539	7415	8264	9100	9918	10733
42½	5730	6622	7509	8368	9216	10044	10869
44¼	6243	7215	8182	9120	10045	10949	11849
44½	6318	7302	8281	9230	10166	11081	11992
46¼	6857	7924	8988	10019	11036	12030	13021
46½	6935	8015	9091	10135	11163	12169	13171
48¼	7499	8667	9831	10961	12074	13163	14248
48½	7581	8762	9939	11081	12207	13308	14405
50¼	8170	9443	10712	11945	13159	14347	15531
50½	8256	9543	10825	12071	13298	14499	15695
52¼	8870	10253	11632	12971	14291	15582	16869
52½	8959	10357	11750	13103	14436	15740	17040
54¼	9598	11096	12589	14040	15470	16869	18263
54½	9692	11204	12712	14177	15621	17033	18441
56¼	10356	11973	13585	15152	16696	18206	19712
56½	10453	12085	13712	15294	16852	18377	19897
58¼	11143	12883	14618	16306	17968	19595	21217
58½	11243	12999	14750	16453	18131	19772	21409
60¼	11958	13827	15690	17502	19288	21035	22777
60½	12062	13947	15826	17655	19456	21219	22976
62¼	12802	14804	16799	18741	20654	22526	24393
62½	12910	14928	16940	18899	20828	22716	24599
64¼	13676	15814	17946	20023	22067	24069	26065
64½	13787	15943	18093	20186	22247	24265	26278
66¼	14578	16858	19132	21347	23527	25662	27792
66½	14693	16991	19283	21515	23713	25865	28012

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 32.—*Continued.*

Moments of Inertia of Four Angles with Equal Legs, Axis X–X.

Moments of Inertia of Four Angles, Axis X–X, Equal Legs.



For Distances Measured from Back to Back.

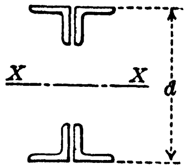
Size.	6″ × 6″										
Thick.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
Area 4 Ls	17.44	20.24	23.00	25.72	28.44	31.12	33.76	36.36	38.92	41.48	44.00
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.⁴										
12½	432	497	560	618	678	735	787	840	891	942	990
14¼	586	675	762	842	924	1004	1077	1151	1223	1293	1362
14½	610	703	793	878	963	1046	1123	1200	1275	1349	1421
16¼	795	917	1035	1147	1260	1370	1472	1575	1675	1773	1869
16½	824	950	1072	1188	1306	1420	1526	1633	1737	1839	1938
18¼	1039	1199	1354	1502	1652	1797	1934	2071	2205	2336	2464
18½	1072	1237	1398	1551	1705	1855	1996	2138	2276	2412	2545
20¼	1317	1521	1720	1910	2101	2288	2464	2640	2812	2982	3147
20½	1354	1564	1769	1964	2161	2353	2535	2716	2894	3069	3239
22¼	1631	1884	2131	2368	2607	2840	3061	3282	3497	3711	3919
22½	1672	1932	2186	2429	2674	2913	3140	3367	3589	3808	4021
24¼	1979	2287	2589	2878	3170	3455	3726	3996	4261	4523	4778
24½	2025	2341	2649	2946	3244	3536	3813	4091	4362	4630	4892
26¼	2362	2731	3092	3440	3789	4131	4458	4784	5102	5417	5725
26½	2412	2790	3159	3513	3871	4220	4554	4887	5212	5535	5850
28¼	2780	3216	3642	4053	4466	4871	5258	5644	6021	6395	6761
28½	2835	3279	3714	4133	4555	4967	5362	5756	6141	6523	6896
30¼	3233	3740	4237	4717	5200	5672	6125	6576	7017	7456	7884
30½	3292	3809	4315	4804	5295	5776	6238	6698	7147	7594	8031
32¼	3721	4306	4879	5433	5990	6535	7060	7581	8092	8599	9095
32½	3784	4379	4962	5526	6093	6648	7181	7712	8232	8748	9253
34¼	4243	4911	5566	6200	6837	7461	8062	8660	9244	9826	10395
34½	4311	4990	5655	6299	6947	7581	8192	8799	9394	9985	10563
36¼	4801	5558	6300	7019	7741	8449	9132	9810	10475	11135	11782
36½	4873	5641	6395	7125	7858	8577	9270	9959	10634	11305	11962
38¼	5393	6244	7079	7889	8702	9500	10269	11034	11783	12528	13257
38½	5470	6333	7180	8001	8826	9635	10416	11192	11952	12708	13448
40¼	6021	6972	7905	8810	9720	10612	11474	12330	13169	14003	14821
40½	6102	7065	8011	8929	9851	10756	11629	12497	13347	14194	15022
42¼	6683	7739	8776	9783	10795	11787	12747	13699	14632	15562	16472
42½	6768	7838	8888	9909	10933	11938	12910	13875	14821	15762	16685
44¼	7380	8548	9694	10808	11926	13024	14087	15141	16174	17203	18211
44½	7470	8651	9812	10939	12072	13183	14259	15326	16372	17414	18435
46¼	8112	9396	10657	11884	13115	14323	15494	16655	17794	18927	20039
46½	8206	9505	10781	12022	13268	14490	15675	16850	18001	19149	20273
48¼	8879	10285	11667	13011	14360	15685	16969	18242	19491	20735	21954
48½	8977	10399	11796	13155	14520	15859	17158	18446	19709	20966	22200
50¼	9681	11215	12722	14190	15663	17108	18511	19902	21266	22625	23957
50½	9783	11334	12857	14341	15829	17291	18709	20115	21493	22867	24214
52¼	10517	12185	13823	15120	17022	18594	20121	21635	23119	24598	26049
52½	10624	12309	13964	15577	17196	18785	20327	21856	23356	24850	26316

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 32.—*Continued.*

Moments of Inertia of Four Angles with Equal Legs, Axis X–X.

Moments of Inertia of Four Angles, Axis X–X, Equal Legs.



For Distances Measured from Back to Back.

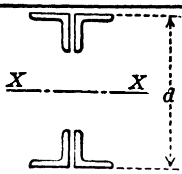
Size.	6″ × 6′										
Thick.	$\frac{3}{8}$″	$\frac{7}{16}$″	$\frac{1}{2}$″	$\frac{9}{16}$″	$\frac{5}{8}$″	$\frac{11}{16}$″	$\frac{3}{4}$′	$\frac{13}{16}$″	$\frac{7}{8}$″	$\frac{15}{16}$″	1″
Area 4[s	17.44	20.24	23.00	25 72	28.44	31.12	33.76	36.36	38.92	41 48	44 00
d″	Moments of Inertia about Axis X–X, for Various Distances Back to Back of Angles, In.^{4}.										
54¼	11389	13196	14971	16701	18438	20143	21799	23440	25050	26654	28228
54½	11500	13325	15118	16865	18619	20341	22013	23671	25297	26917	28507
56¼	12295	14247	16164	18034	19911	21753	23544	25318	27058	28793	30495
56½	12411	14381	16317	18205	20099	21959	23767	25558	27315	29066	30785
58¼	13236	15338	17404	19419	21440	23426	25357	27269	29145	31015	32851
58½	13356	15478	17562	19596	21636	23639	25588	27518	29411	31299	33151
60¼	14212	16470	18689	20855	23027	25161	27237	29292	31309	33321	35294
60½	14337	16615	18853	21038	23230	25382	27476	29550	31585	33614	35605
62¼	15223	17643	20021	22342	24671	26958	29184	31388	33551	35709	37825
62½	15352	17792	20191	22532	24880	27187	29432	31655	33837	36013	38148
64¼	16269	18856	21398	23881	26371	28817	31199	33557	35872	38179	40445
64½	16402	19010	21574	24077	26588	29054	31456	33833	36167	38494	40778
66¼	17350	20109	22822	25471	28128	30739	33282	35799	38269	40733	43152
66½	17488	20269	23003	25673	28352	30984	33547	36084	38575	41058	43496
63¼	18466	21403	24291	27113	29943	32723	35432	38113	40745	43370	45947
68½	18608	21568	24478	27322	30173	32975	35706	38407	41060	43706	46303
70¼	19616	22738	25807	28806	31814	34769	37650	40500	43299	46090	48830
70½	19762	22907	25999	29022	32052	35029	37932	40803	43623	46436	49197
72¼	20801	24113	27368	30551	33742	36877	39935	42960	45930	48893	51802
72½	20952	24287	27567	30773	33987	37145	40225	43272	46264	49249	52179
74½	22177	25708	29180	32575	35979	39324	42587	45814	48983	52145	55250
76½	23436	27169	30839	34429	38027	41564	45015	48428	51780	55124	58408
78½	24731	28670	32544	36334	40133	43867	47512	51115	54655	58186	61654
80½	26060	30212	34295	38291	42296	46232	50075	53875	57607	61331	64989
82½	27424	31794	36093	40299	44515	48660	52707	56707	60638	64559	68411
84½	28823	33417	37936	42359	46792	51149	55405	59612	63746	67870	71921
86½	30257	35080	39825	44470	49125	53701	58172	62590	66932	71264	75520
88½	31726	36784	41760	46633	51515	56315	61005	65641	70196	74741	79206
90½	33230	38528	43742	48847	53962	58992	63907	68764	73537	78301	82980
92½	34768	40313	45769	51112	56466	61730	66876	71960	76957	81943	86843
94½	36342	42138	47842	53429	59026	64531	69912	75229	80454	85669	90793
96½	37950	44004	49961	55797	61644	67394	73016	78571	84029	89478	94831
98½	39593	45910	52126	58217	64319	70319	76187	81985	87682	93369	98958
100½	41271	47857	54338	60689	67050	73307	79426	85472	91413	97344	103172
102½	42984	49844	56595	63211	69838	76357	82733	89031	95222	101401	107474
104½	44732	51872	58898	65785	72683	79469	86107	92664	99109	105542	111865
106½	46515	53940	61247	68411	75585	82643	89548	96369	103074	109765	116343
108½	48332	56049	63643	71088	78544	85879	93057	100147	107116	114072	120909
110½	50185	58198	66084	73817	81560	89178	96634	103997	111236	118461	125563
112½	52072	60387	68571	76597	84633	92539	100278	107920	115434	122934	130306

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 32.—*Continued.*

Moments of Inertia of Four Angles with Equal Legs, Axis X-X.

Moments of Inertia of Four Angles, Axis X-X, Equal Legs.



For Distances Measured from Back to Back.

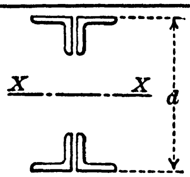
Size.	8″ × 8″										
Thick.	½″	$\frac{9}{16}$″	⅝″	$\frac{11}{16}$″	¾″	$\frac{13}{16}$″	⅞″	$\frac{15}{16}$″	1″	$1\frac{1}{16}$″	1⅛″
Area 4 ∟s	31.00	34.72	38.44	42.12	45.76	49.36	52.92	56.48	60.00	63.48	66.92
d″	Moments of Inertia About Axis X-X for Various Distances Back to Back of Angles, In.^{4}.										
16½	1333	1483	1631	1775	1910	2046	2179	2310	2430	2554	2674
18¼	1686	1877	2065	2249	2423	2598	2769	2937	3094	3254	3409
18½	1740	1937	2132	2322	2502	2683	2860	3034	3196	3361	3523
20¼	2146	2391	2634	2871	3095	3321	3542	3760	3964	4172	4375
20½	2208	2461	2710	2954	3186	3419	3646	3871	4082	4296	4505
22¼	2669	2976	3279	3576	3859	4143	4421	4696	4955	5218	5475
22½	2739	3054	3365	3670	3961	4253	4538	4821	5087	5357	5621
24¼	3254	3630	4001	4366	4714	5064	5406	5745	6066	6390	6708
24½	3332	3716	4097	4471	4828	5186	5536	5884	6213	6546	6871
26¼	3901	4353	4801	5240	5661	6083	6497	6907	7296	7690	8075
26½	3987	4448	4906	5355	5786	6217	6640	7060	7458	7861	8255
28¼	4610	5145	5677	6198	6699	7201	7693	8182	8647	9116	9576
28½	4703	5249	5792	6324	6835	7348	7850	8349	8824	9303	9773
30¼	5381	6008	6630	7241	7829	8418	8996	9569	10117	10669	11211
30½	5482	6120	6754	7377	7977	8577	9166	9751	10310	10872	11425
32¼	6214	6939	7659	8367	9050	9733	10404	11070	11708	12350	12980
32½	6323	7060	7794	8514	9209	9904	10587	11266	11915	12569	13210
34¼	7109	7940	8766	9578	10363	11147	11918	12684	13419	14157	14882
34½	7225	8070	8910	9736	10534	11331	12114	12893	13641	14392	15129
36¼	8066	9010	9950	10873	11768	12660	13538	14410	15249	16091	16919
36½	8190	9149	10103	11041	11950	12856	13748	14634	15486	16342	17183
38¼	9085	10150	11210	12253	13264	14272	15263	16250	17200	18152	19089
38½	9217	10298	11373	12431	13457	14480	15487	16488	17452	18419	19369
40¼	10166	11360	12547	14717	14851	15982	17095	18202	19270	20340	21393
40½	10306	11516	12720	13905	15056	16203	17331	18454	19538	20623	21690
42¼	11309	12638	13962	15264	16530	17791	19032	20268	21461	22656	23831
42½	11456	12803	14144	15464	16746	18024	19282	20534	21743	22954	24145
44¼	12514	13987	15453	16897	18300	19699	21076	22446	23772	25098	26402
44½	12669	14160	15645	17107	18528	19944	21338	22726	24069	25412	26733
46¼	13781	15404	17021	18613	20162	21705	23225	24738	26202	27667	29108
46½	13944	15586	17222	18833	20401	21963	23501	25032	26514	27997	29456
48¼	15110	16891	18666	20414	22116	23811	25480	27142	28753	30363	31947
48½	15280	17082	18877	20645	22366	24081	25769	27450	29080	30709	32312
50¼	16501	18448	20387	22299	24161	26014	27840	29659	31423	33186	34921
50½	16679	18647	20608	22540	24423	26291	28143	29982	31766	33548	35302
52¼	17954	20074	22186	24268	26297	28317	30307	32290	34214	36136	38028
52½	18140	20282	22416	24520	26571	28612	30623	32626	34571	36513	38426
54¼	19469	21769	24061	26321	28525	30718	32879	35033	37125	39212	41269
54½	19663	21986	24301	26584	28810	31026	33208	35384	37497	39606	41684
56¼	21046	23534	26014	28459	30845	33219	35578	37889	40155	42416	44644
56½	21247	23759	26263	28732	31141	33538	35900	38254	40542	42826	45075

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 32.—*Continued.*

Moments of Inertia of Four Angles with Equal Legs, Axis X–X.

Moments of Inertia of Four Angles, Axis X–X, Equal Legs.



For Distances Measured from Back to Back.

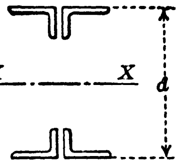
Size.	8″ × 8″										
Thick.	½″	9/16″	⅝″	11/16″	¾″	13/16″	⅞″	15/16″	1″	1 1/16″	1⅛″
Area 4 ∟s	31.00	34.72	38.44	42.12	45.76	49.36	52.92	56.48	60.00	63.48	66 92
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.⁴.										
58¼	22685	25368	28043	30680	33256	35817	38342	40858	43306	45747	48152
58½	22894	25602	28302	30964	33564	36149	38697	41237	43708	46172	48601
60¼	24386	27272	30149	32986	35759	38515	41232	43940	46576	49205	51795
60½	24603	27515	30418	33281	36078	38859	41600	44333	46994	49646	52260
62¼	26149	29245	32332	35377	38353	41311	44228	47135	49967	52789	55571
62½	26376	29497	32610	35681	38683	41668	44609	47543	50399	53247	56053
64¼	27974	31288	34592	37851	41038	44206	47329	50443	53478	56501	59481
64½	28206	31548	34880	38167	41381	44575	47724	50865	53925	56974	59980
66¼	29861	33400	36929	40410	43816	47200	50537	53864	57108	60340	63525
66½	30101	33669	37226	40736	44169	47581	50945	54300	57570	60829	64040
68¼	31810	35581	39342	43053	46684	50292	53850	57398	60859	64305	67703
68½	32058	35859	39650	43389	47049	50686	54272	57848	61336	64810	68235
70¼	33821	37832	41833	45780	49645	53483	57269	61045	64729	68398	72015
70½	34076	38118	42150	46127	50021	53889	57704	61509	65222	68919	72563
72¼	35894	40152	44400	48592	52696	56773	60794	64805	68720	72617	76460
72½	36157	40447	44727	48949	53084	57191	61242	65283	69227	73154	77025
74½	38300	42846	47381	51856	56239	60592	64886	69170	73353	77516	81621
76½	40505	45314	50111	54846	59485	64092	68636	73170	77598	82006	86351
78½	42771	47851	52919	57921	62823	67690	72492	77283	81964	86622	91215
80½	45100	50458	55804	61080	66252	71387	76453	81509	86450	91365	96213
82½	47491	53134	58765	64323	69773	75183	80521	85847	91055	96235	101344
84½	49943	55880	61803	67651	73385	79077	84694	90299	95781	101233	106609
86½	52458	58695	64919	71062	77089	83071	88973	94864	100626	106357	112008
88½	55035	61579	68111	74558	80884	87163	93398	99541	105592	111608	117541
90½	57674	64533	71380	78139	84771	91353	97849	104332	110678	116986	123208
92½	60374	67557	74725	81803	88749	95643	102446	109236	115883	122491	129009
94½	63137	70650	78148	85552	92819	100031	107148	114252	121209	128123	134943
96½	65962	73812	81648	89385	96981	104518	111956	119382	126654	133882	141011
98½	68848	77044	85224	93302	101234	109103	116871	124624	132220	139767	147214
100½	71797	80345	88877	97303	105578	113787	121891	129980	137906	145780	153550
102½	74808	83715	92608	101389	110014	118570	127016	135448	143711	151920	160019
104½	77881	87155	96415	105559	114542	123452	132248	141029	149637	158187	166623
106½	81015	90665	100299	109813	119161	128432	137587	146723	155682	164581	173361
108½	84212	94244	104260	114151	123871	133512	143029	152531	161848	171101	180232
110½	87471	97892	108297	118574	128673	138689	148578	158451	168134	177749	187237
112½	90792	101610	112412	123081	133567	143966	154233	164484	174539	184523	194376
114½	94174	105397	116603	127672	138552	149341	159994	170630	181065	191425	201649
116½	97619	109254	120872	132347	143628	154815	165861	176890	187710	198454	209056
118½	101126	113180	125217	137107	148796	160388	171833	183262	194476	205609	216596
120½	104694	117176	129639	141950	154056	166060	177912	189747	201362	212891	224270

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 33.

MOMENTS OF INERTIA OF FOUR ANGLES WITH UNEQUAL LEGS, AXIS X–X. LONG LEGS TURNED OUT.

Moments of Inertia of Four Angles, Axis X–X, Long Legs Turned Out.



For Distances Measured from Back to Back.

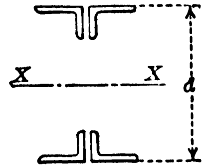
Size.	3″ × 2½″, Long Legs Out.						3½″×2½″, Long Legs Out.							
Thick	¼″	5/16″	⅜″	7/16″	½″	9/16″	¼″	5/16″	⅜″	7/16″	½″	9/16″	⅝″	11/16″
Area 4 Ls	5.24	6.48	7.68	8.88	10.00	11.12	5.76	7.12	8.44	9.72	11.00	12.24	13.44	14.60
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.^{4}.													
5½	26	31	36	41	45	49	30	35	41	47	52	56	60	64
5¾	29	35	41	46	51	55	34	40	46	53	59	62	67	72
6	32	39	45	52	57	61	37	44	52	59	65	69	74	79
6¼	35	43	50	57	63	67	41	49	57	65	72	76	82	88
6½	38	47	55	62	69	74	44	53	62	70	79	84	90	97
6¾	41	51	59	68	75	81	48	58	68	76	85	92	99	106
7	45	55	64	73	81	89	51	62	73	82	92	100	108	116
7¼	49	60	69	79	87	96	55	67	79	89	99	109	118	126
7½	53	65	75	86	95	104	60	73	85	97	108	118	127	137
7¾	57	70	81	93	103	113	64	78	92	104	116	127	138	148
8	61	75	87	100	111	122	69	84	99	112	125	137	148	159
8¼	66	81	94	107	119	131	74	90	106	120	134	147	159	171
8½	71	86	100	115	128	140	79	97	113	129	144	158	171	184
8¾	75	92	107	123	137	150	85	103	121	138	154	169	183	197
9	80	98	114	131	146	160	90	110	129	147	164	180	195	210
9¼	85	104	122	139	155	171	96	117	137	156	175	192	208	224
9½	91	111	129	148	165	182	102	124	146	166	186	204	221	238
9¾	96	118	137	157	175	193	108	131	154	176	197	216	235	253
10	102	125	145	167	186	205	114	139	163	186	209	229	249	268
10¼	107	132	154	176	197	217	121	147	173	197	221	242	264	284
10½	113	139	162	186	208	229	127	155	182	208	233	256	279	300
10¾	120	146	171	196	219	241	134	163	192	219	246	270	294	316
11	126	154	180	207	231	254	141	172	202	231	259	285	310	334
11¼	132	162	190	218	243	268	148	181	212	243	272	299	326	351
11½	139	170	199	229	255	281	155	190	223	256	286	315	342	369
11¾	146	178	209	240	268	295	163	199	234	267	300	330	359	387
12	152	187	219	251	281	310	170	208	245	280	314	346	377	406
12¼	159	196	229	263	294	325	178	218	256	293	329	362	395	426
12½	167	205	240	275	308	340	186	228	268	306	344	379	413	445
12¾	174	214	250	288	322	355	195	238	280	320	360	396	432	465
13	182	223	261	301	336	371	203	248	292	334	375	414	451	486
13¼	189	233	273	314	350	387	212	259	305	349	392	431	470	507
13½	197	242	284	327	365	403	220	270	317	363	408	450	490	529
13¾	205	252	296	340	380	420	229	281	330	378	425	468	511	551
14	214	262	308	354	396	437	238	292	344	393	442	487	531	574
14¼	222	273	320	368	412	455	248	303	357	409	460	507	553	597
14½	231	283	333	382	428	473	257	315	371	424	477	526	574	620
14¾	239	294	345	397	444	491	267	327	385	441	495	547	596	644
15	248	305	358	412	461	509	277	339	399	457	514	560	619	668
15¼	257	316	371	427	478	528	287	351	414	474	533	588	642	693
15½	266	327	385	443	495	547	297	364	429	491	552	609	665	718
15¾	276	339	398	458	513	567	307	376	444	508	572	631	689	744

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 33.—*Continued.*

Moments of Inertia of Four Angles with Unequal Legs, Axis X–X.

Long Legs Turned Out.



Moments of Inertia of Four Angles, Axis X–X, Long Legs Turned Out.

For Distances Measured from Back to Back.

Size.	4″ × 3″, Long Legs Turned Out.														
Thick.	1/4″	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	Thick.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″
Area 4 ∟s	6.76	8.36	9.92	11.48	13.00	14.48	15.92	Area 4 ∟s	9.92	11.48	13.00	14.48	15.92	17.36	18.76
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.^{4}.														
6½	48	58	68	78	86	94	102	16	525	604	678	751	821	890	954
6¾	52	63	84	85	94	103	111	16¼	543	625	702	777	849	921	987
7	57	69	81	92	102	112	122	16½	561	646	725	804	879	953	1023
7¼	62	75	88	100	111	122	132	16¾	580	667	750	831	908	983	1055
7½	67	81	95	109	121	132	144	18¼	699	804	904	1002	1096	1190	1276
7¾	72	88	103	117	130	143	155	18½	719	828	931	1032	1129	1226	1315
8	77	94	111	126	140	154	167	20¼	874	1007	1133	1256	1375	1493	1603
8¼	83	101	119	136	151	166	180	20½	897	1034	1163	1290	1412	1533	1646
8½	89	108	127	145	162	178	193	22¼	1069	1233	1388	1539	1686	1831	1967
8¾	95	116	136	155	173	191	207	22½	1095	1262	1421	1577	1727	1876	2015
9	101	124	145	166	185	204	221	24¼	1284	1481	1668	1851	2028	2204	2368
9¼	107	131	154	177	197	217	236	24½	1313	1514	1705	1892	2073	2253	2421
9½	114	140	164	188	209	231	251	26¼	1519	1753	1975	2192	2402	2611	2808
9¾	121	148	174	199	222	245	267	26½	1550	1788	2015	2237	2451	2664	2865
10	128	157	184	211	236	260	283	28¼	1774	2047	2308	2562	2809	3053	3284
10¼	135	166	195	223	249	275	300	28½	1808	2085	2351	2611	2862	3111	3347
10½	143	175	206	236	264	291	317	30¼	2049	2364	2666	2961	3247	3530	3799
10¾	151	185	217	249	278	307	335	30½	2085	2406	2713	3013	3303	3592	3865
11	159	194	229	262	293	324	353	32¼	2344	2705	3051	3389	3716	4042	4350
11¼	167	204	241	276	309	341	371	32½	2382	2749	3101	3445	3777	4108	4422
11½	175	215	253	290	324	358	391	34¼	2658	3068	3462	3846	4218	4588	4940
11¾	184	225	265	304	341	376	410	34½	2699	3115	3515	3905	4283	4659	5016
12	192	236	278	319	357	395	430	36¼	2992	3455	3898	4332	4751	5169	5566
12¼	201	247	291	334	374	414	451	36½	3035	3504	3955	4395	4820	5244	5647
12½	211	259	305	350	392	433	472	38¼	3346	3864	4361	4847	5317	5785	6231
12¾	220	270	318	366	409	453	494	38½	3392	3917	4421	4913	5390	5864	6316
13	230	282	332	382	428	473	516	40¼	3720	4296	4850	5390	5914	6435	6932
13¼	240	294	347	398	446	494	539	40½	3768	4352	4912	5460	5991	6519	7023
13½	250	307	361	415	465	515	562	42¼	4114	4751	5364	5963	6543	7120	7672
13¾	260	319	376	432	485	536	585	42½	4164	4810	5430	6037	6624	7209	7767
14	270	332	391	450	505	558	610	44¼	4527	5229	5905	6565	7204	7840	8449
14¼	281	345	407	468	525	581	634	44½	4580	5291	5974	6642	7289	7933	8548
14½	292	359	423	486	546	604	659	46¼	4961	5730	6472	7195	7896	8595	9263
14¾	303	372	439	505	567	627	685	46½	5016	5795	6544	7276	7986	8692	9367
15	314	386	456	524	588	651	711	48¼	5414	6254	7064	7855	8621	9384	10115
15¼	326	401	472	543	610	675	738	48½	5472	6322	7140	7939	8714	9486	10224
15½	338	415	490	563	632	700	765	50¼	5887	6801	7683	8543	9377	10208	11004
15¾	350	430	507	583	655	725	792	50½	5948	6871	7762	8631	9475	10314	11118

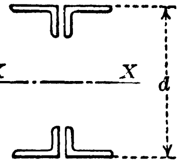
Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 33.—*Continued.*

Moments of Inertia of Four Angles with Unequal Legs, Axis X–X.

Long Legs Turned Out.

Moments of Inertia of Four Angles, Axis X–X, Long Legs Turned Out.



For Distances Measured from Back to Back.

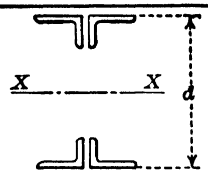
Size.	5″ × 3″, Long Legs Turned Out.													
Thick.	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	Thick.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″
Area 4 ∟s	9.60	11.44	13.24	15.00	16.72	18.44	20.12	Area 4 ∟s	11.44	13.24	15.00	16.72	18.44	20.12
d″	Moments of Inertia About Axis X-X for Various Distances Back to Back of Angles, In.⁴													
6½	73	83	93	104	114	123	132							
6¾	78	90	102	114	125	135	145							
7	83	98	111	124	136	147	158	18¼	820	942	1062	1179	1290	1401
7¼	90	106	120	134	148	159	171	18½	845	970	1094	1214	1329	1443
7½	97	115	130	145	160	173	186	20¼	1024	1178	1329	1475	1616	1755
7¾	105	124	140	157	172	187	201	20½	1052	1209	1364	1514	1659	1802
8	113	133	151	169	186	201	217	22¼	1251	1440	1625	1804	1978	2150
8¼	121	142	162	181	200	216	233	22½	1282	1475	1664	1848	2026	2202
8½	129	152	173	194	214	232	250	24¼	1501	1728	1951	2167	2377	2585
8¾	138	163	185	207	229	248	267	24½	1534	1766	1994	2215	2430	2642
9	147	173	197	221	244	265	286	26¼	1774	2043	2307	2564	2813	3060
9¼	156	184	210	236	260	282	304	26½	1810	2085	2354	2615	2871	3122
9½	166	196	223	250	276	300	334	28¼	2070	2385	2694	2994	3286	3575
9¾	176	208	237	265	293	318	344	28½	2109	2429	2744	3049	3348	3642
10	186	220	251	281	310	337	365	30¼	2389	2753	3110	3457	3796	4130
10¼	197	232	265	297	328	357	386	30½	2430	2801	3164	3517	3863	4203
10½	207	245	280	314	346	377	408	32¼	2730	3147	3556	3954	4343	4726
10¾	219	258	295	331	366	398	431	32½	2774	3198	3614	4018	4414	4803
11	230	272	311	349	385	420	454	34¼	3094	3568	4032	4484	4927	5362
11¼	242	286	327	367	405	442	478	34½	3142	3623	4094	4552	5002	5444
11½	254	300	342	385	426	464	502	36¼	3482	4016	4539	5047	5547	6038
11¾	266	315	360	404	447	487	527	36½	3532	4073	4604	5120	5627	6126
12	277	330	377	424	469	511	553	38¼	3892	4489	5075	5645	6205	6755
12¼	292	345	395	444	491	535	579	38½	3945	4551	5144	5721	6289	6847
12½	305	361	413	464	513	560	606	40¼	4325	4990	5641	6275	6899	7512
12¾	318	377	432	485	537	585	634	40½	4381	5054	5714	6356	6988	7609
13	332	393	451	506	560	611	662	42¼	4781	5517	6237	6939	7630	8309
13¼	346	410	470	528	585	638	691	42½	4839	5584	6314	7024	7724	8411
13½	361	427	490	550	609	665	721	44¼	5259	6070	6864	7636	8398	9146
13¾	375	444	510	573	634	693	751	44½	5321	6141	6944	7726	8497	9253
14	390	462	530	596	660	721	782	46¼	5761	6650	7520	8367	9203	10023
14¼	406	480	551	620	687	750	813	46½	5825	6724	7604	8461	9306	10136
14½	421	499	573	644	713	779	845	48¼	6286	7256	8206	9132	10045	10941
14¾	437	518	595	668	741	809	878	48½	6353	7334	8294	9229	10153	11058
15	453	537	617	694	769	840	911	50¼	6833	7889	8922	9929	10923	11899
15¼	470	557	639	719	797	871	945	50½	6903	7970	9014	10031	11036	12021
15½	487	577	662	745	826	903	979	52¼	7403	8548	9668	10760	11839	12897
15¾	504	597	686	772	855	935	1015	52½	7476	8632	9764	10866	11956	13024
16	521	618	710	799	885	968	1050	54¼	7996	9234	10445	11625	12791	13935
16¼	539	639	734	826	916	1002	1087	54½	8072	9321	10544	11735	12913	14067
16½	557	660	759	854	947	1036	1124	56¼	8612	9946	11251	12523	13781	15014
16¾	575	682	784	882	978	1070	1162	56½	8691	10037	11354	12637	13907	15152

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 33.—*Continued.*

MOMENTS OF INERTIA OF FOUR ANGLES WITH UNEQUAL LEGS, AXIS X–X.
LONG LEGS TURNED OUT.

Moments of Inertia of Four Angles, Axis X–X, Long Legs Turned Out



For Distances Measured from Back to Back.

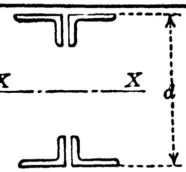
Size.	5″ × 3½″, Long Legs Turned Out.															
Thick.	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	Thick.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″
Area 4 ⌊s	10.24	12.20	14.12	16.00	17.88	19.68	21.48	23.24	Area 4 ⌊s	12.20	14.12	16.00	17.88	19.68	21.48	23.24
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.⁴															
7½	98	115	131	145	160	174	187	198								
7¾	105	124	141	157	173	188	202	214								
8	113	133	152	169	186	202	218	231	20¼	1060	1221	1375	1530	1676	1821	1957
8¼	121	143	163	182	200	217	235	249	20½	1088	1254	1412	1571	1721	1871	2011
8½	130	153	175	195	215	233	252	268	22¼	1298	1497	1686	1876	2057	2236	2405
8¾	139	163	187	208	230	249	270	287	22½	1330	1533	1727	1922	2107	2291	2464
9	148	174	200	222	246	267	288	307	24¼	1561	1800	2029	2259	2477	2694	2899
9¼	158	186	213	237	262	284	308	328	24½	1595	1840	2074	2309	2532	2754	2964
9½	167	197	226	252	279	303	328	349	26¼	1848	2132	2404	2677	2937	3194	3439
9¾	178	209	240	268	296	322	348	371	26½	1886	2175	2453	2732	2997	3260	3510
10	188	222	254	284	314	341	370	394	28¼	2159	2492	2810	3131	3435	3738	4026
10¼	199	235	269	300	332	362	392	418	28½	2200	2539	2864	3190	3501	3809	4102
10½	210	248	284	318	351	382	414	442	30¼	2495	2880	3249	3621	3974	4325	4659
10¾	221	261	300	335	371	404	438	467	30½	2539	2930	3306	3684	4044	4401	4741
11	233	275	316	353	391	426	462	493	32¼	2856	3296	3720	4146	4551	4954	5339
11¼	245	290	332	372	412	449	486	519	32½	2902	3350	3781	4214	4626	5036	5427
11½	258	304	349	391	433	472	512	547	34¼	3240	3741	4223	4707	5168	5627	6065
11¾	270	320	367	411	455	496	538	574	34½	3290	3798	4288	4780	5248	5714	6159
12	284	335	385	431	477	520	564	603	36¼	3649	4214	4758	5304	5825	6342	6838
12¼	297	351	403	451	500	546	592	633	36½	3702	4275	4827	5381	5909	6435	6938
12½	311	367	422	472	524	571	620	663	38¼	4083	4715	5325	5937	6520	7101	7657
12¾	325	384	441	494	548	598	648	694	38½	4139	4779	5398	6019	6610	7199	7763
13	339	401	460	516	573	625	678	725	40¼	4541	5244	5924	6606	7255	7902	8523
13¼	354	418	481	539	598	652	708	758	40½	4600	5312	6001	6692	7350	8005	8634
13½	369	436	501	562	623	681	738	791	42¼	5023	5802	6555	7310	8030	8747	9435
13¾	384	454	522	586	650	709	770	824	42½	5085	5873	6636	7400	8129	8855	9552
14	399	473	543	610	677	739	802	859	44¼	5530	6388	7217	8050	8843	9634	10393
14¼	415	492	565	634	704	769	835	894	44½	5595	6463	7303	8145	8948	9748	10517
14½	432	511	587	659	732	800	868	930	46¼	6061	7002	7912	8826	9697	10564	11399
14¾	448	531	610	685	761	831	902	967	46½	6129	7080	8001	8925	9806	10683	11527
15	465	551	633	711	790	863	938	1004	48¼	6616	7644	8639	9637	10589	11537	12450
15¼	482	571	657	738	819	895	972	1042	48½	6687	7726	8732	9741	10703	11662	12585
15½	500	592	681	765	849	929	1008	1081	50¼	7196	8315	9398	10485	11521	12554	13548
15¾	518	613	705	792	880	962	1045	1121	50½	7270	8400	9495	10593	11640	12684	13688
16	536	635	730	820	912	997	1082	1161	52¼	7800	9013	10189	11368	12492	13613	14693
16¼	554	657	756	849	943	1032	1120	1202	52½	7878	9103	12090	11481	12616	13748	14839
16½	573	679	781	878	976	1067	1159	1244	54¼	8429	9740	11012	12287	13503	14715	15884
16¾	592	702	808	908	1009	1104	1199	1286	54½	8509	9833	11117	12404	13632	14856	16035
18	693	821	945	1063	1182	1294	1406	1510	56¼	9082	10496	11867	13241	14553	15860	17121
18¼	714	846	974	1096	1219	1334	1449	1556	56½	9165	10592	11976	13363	14687	16006	17279
18½	735	872	1004	1129	1256	1374	1493	1604	58¼	9759	11279	12754	14232	15642	17048	18405
18¾	757	897	1033	1163	1293	1415	1538	1652	58½	9846	11379	12867	14358	15781	17199	18569

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 33.—*Continued.*

MOMENTS OF INERTIA OF FOUR ANGLES WITH UNEQUAL LEGS, AXIS X-X. LONG LEGS TURNED OUT.

Moments of Inertia of Four Angles, Axis X-X, Long Legs Turned Out.



For Distances Measured from Back to Back.

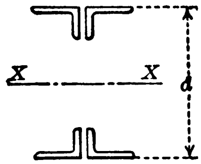
Size.	6″ × 4″, Long Legs Turneu Out.										
Thick.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
Area 4 ⌊s	14.44	16 72	19.00	21.24	23 44	25.60	27.76	29.88	31.92	34.00	36.00
d″	Moments of Inertia About Axis X-X for Various Distances Back to Back of Angles, In.⁴.										
8½	178	203	227	251	273	293	314	333	352	370	385
10¼	273	312	350	387	423	455	489	521	551	581	606
10½	288	330	370	409	448	482	517	552	583	615	642
12¼	408	468	526	583	639	689	741	791	839	886	927
12½	427	490	551	611	669	722	777	829	879	929	972
14¼	572	658	740	822	901	974	1049	1122	1190	1259	1320
14½	595	684	770	855	937	1013	1092	1167	1238	1310	1374
16¼	765	881	992	1103	1210	1310	1413	1512	1605	1700	1784
16½	791	911	1027	1141	1252	1356	1462	1564	1662	1760	1848
18¼	987	1137	1282	1426	1566	1698	1831	1961	2084	2209	2321
18½	1017	1171	1321	1170	1614	1750	1888	2022	2149	2277	2393
20¼	1238	1427	1611	1792	1969	2136	2306	2471	2627	2786	2930
20½	1271	1465	1654	1841	2023	2195	2369	2539	2700	2863	3011
22¼	1518	1750	1977	2201	2419	2626	2836	3040	3234	3431	3611
22½	1555	1793	2025	2255	2478	2691	2906	3115	3315	3516	3701
24¼	1826	2107	2381	2652	2916	3167	3421	3669	3905	4144	4363
24½	1867	2154	2434	2711	2981	3238	3498	3752	3993	4238	4463
26¼	2164	2497	2823	3145	3459	3759	4062	4358	4639	4925	5188
26½	2208	2548	2881	3210	3530	3837	4146	4448	4736	5027	5296
28¼	2530	2920	3303	3681	4050	4402	4759	5106	5438	5775	6085
28½	2578	2976	3366	3751	4127	4486	4850	5204	5542	5885	6202
30¼	2925	3377	3821	4259	4687	5097	5511	5914	6300	6692	7054
30½	2977	3437	3889	4335	4770	5187	5609	6020	6412	6810	7180
32¼	3349	3868	4377	4880	5371	5842	6318	6782	7226	7677	8094
32½	3404	3931	4450	4961	5460	5939	6423	6895	7346	7804	8230
34¼	3802	4391	4971	5544	6102	6639	7181	7710	8216	8730	9207
34½	3861	4459	5048	5629	6197	6743	7293	7830	8344	8865	9351
36¼	4284	4949	5604	6249	6880	7488	8100	8698	9269	9851	10392
36½	4346	5021	5685	6341	6981	7597	8219	8825	9406	9995	10545
38¼	4795	5539	6274	6998	7705	8387	9074	9745	10387	11040	11649
38½	4861	5616	6360	7094	7811	8503	9200	9880	10531	11192	11811
40¼	5334	6164	6982	7788	8577	9337	10104	10852	11568	12297	12978
40½	5404	6244	7073	7890	8689	9460	10236	10995	11720	12458	13149
42¼	5903	6821	7728	8622	9495	10339	11189	12019	12813	13622	14378
42½	5976	6906	7824	8729	9613	10468	11328	12169	12974	13791	14558
44¼	6500	7512	8512	9497	10461	11392	12329	13245	14122	15015	15851
44½	6577	7601	8613	9610	10585	11527	12476	13403	14291	15193	16040
46¼	7127	8237	9334	10416	11473	12496	13526	14532	15495	16476	17396
46½	7207	8330	9440	10533	11603	12638	13679	14697	15671	16662	17594
48¼	7787	8995	10194	11376	12533	13651	14777	15878	16932	18005	19013
48½	7866	9092	10305	11499	12668	13800	14938	16050	17116	18199	19220

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 33.—*Continued.*

Moments of Inertia of Four Angles with Unequal Legs, Axis X–X. Long Legs Turned Out.

Moments of Inertia of Four Angles, Axis X–X, Long Legs Turned Out.



For Distances Measured from Back to Back.

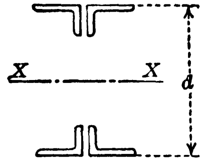
Size.	6″ × 4″, Long Legs Turned Out.										
Thick.	⅜″	7/16″	½″	9/16″	⅝″	11/16″	¾″	13/16″	⅞″	15/16″	1″
Area 4∟s	14.44	16.72	19.00	21.24	23.44	25.60	27.76	29.88	31.92	34.00	36.00
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.⁴										
50¼	8466	9786	11093	12379	13639	14858	16085	17284	18433	19602	20701
50½	8553	9887	11207	12508	13780	15012	16252	17464	18625	19805	20917
52¼	9179	10611	12029	13425	14792	16116	17447	18749	19997	21267	22462
52½	9270	10716	12148	13559	14939	16277	17622	18937	20197	21478	22687
54¼	9921	11469	13003	14513	15992	17425	18866	20275	21626	23000	24295
54½	10015	11579	13127	14652	16145	17592	19047	20470	21833	23220	24529
56¼	10691	12361	14015	15644	17238	18785	20339	21860	23318	24801	26200
56½	10789	12475	14144	15788	17397	18958	20527	22062	23533	25029	26443
58¼	11491	13286	15065	16817	18532	20196	21869	23505	25074	26669	28176
58½	11593	13404	15199	16967	18697	20376	22064	23715	25297	26907	28429
60¼	12319	14244	16153	18032	19873	21659	23453	25209	26894	28606	30225
60½	12425	14367	16292	18187	20043	21845	23655	25427	27125	28852	30486
62¼	13176	15236	17279	19291	21260	23172	25094	26974	28778	30611	32346
62½	13286	15363	17423	19451	21437	23365	25303	27199	29017	30866	32616
64¼	14063	16262	18443	20591	22694	24737	26790	28798	30725	32684	34539
64½	14175	16392	18592	20757	22877	24937	27006	29030	30972	32947	34818
66¼	14978	17321	19646	21934	24175	26353	28541	30682	32736	34825	36803
66½	15094	17455	19799	22105	24364	26559	28764	30922	32991	35097	37092
68¼	15922	18413	20886	23320	25703	28021	30348	32625	34811	37034	39140
68½	16042	18552	21043	23496	25898	28233	30578	32873	35074	37314	39437
70¼	16894	19539	22164	24747	27278	29739	32210	34629	36950	39311	41549
70½	17018	19682	22326	24929	27478	29958	32447	34885	37221	39600	41855
72¼	17896	20698	23480	26218	28900	31509	34128	36692	39153	41656	44030
72½	18023	20845	23647	26405	29106	31734	34372	36955	39432	41953	44345
74½	19057	22042	25006	27923	30781	33561	36352	39086	41707	44375	46907
76½	20121	23272	26403	29484	32502	35440	38388	41276	44045	46864	49540
78½	21212	24536	27838	31087	34270	37370	40480	43526	46447	49422	52246
80½	22333	25833	29311	32733	36086	39350	42627	45836	48914	52047	55024
82½	23483	27164	30822	34421	37948	41383	44829	48205	51444	54741	57874
84½	24662	28528	32370	36151	39857	43466	47087	50634	54037	57502	60795
86½	25869	29925	33957	37925	41812	45600	49401	53123	56695	60332	63789
88½	27105	31356	35582	39740	43815	47786	51770	55672	59417	63229	66855
90½	28371	32821	37245	41598	45865	50023	54194	58281	62202	66195	69993
92½	29665	34318	38946	43499	47961	52311	56674	60949	65051	69228	73202
94½	30988	35850	40685	45442	50105	54651	59210	63677	67964	72330	76484
96½	32340	37414	42462	47427	52295	57041	61801	66465	70941	75499	79838
98½	33720	39012	44277	49455	54532	59483	64448	69312	73982	78736	83264
100½	35130	40644	46129	51526	56816	61976	67150	72220	77086	82402	87761
102½	36569	43309	48020	53639	59147	64520	69908	75187	80254	85415	90331
104½	38036	44007	49949	55794	61524	67115	72721	78214	83487	88857	93973

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 33.—*Continued.*

MOMENTS OF INERTIA OF FOUR ANGLES WITH UNEQUAL LEGS, AXIS X–X. LONG LEGS TURNED OUT.

Moments of Inertia of Four Angles, Axis X-X, Long Legs Turned Out.



For Distances Measured from Back to Back.

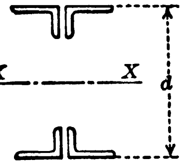
Size.	8″ × 6″, Long Legs Turned Out.									
Thick.	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
Area 4 ∟s	23.72	27.00	30.24	33.44	36.60	39.76	42.88	45.92	49.00	52.00
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.⁴.									
12½	624	704	778	853	926	997	1062	1128	1193	1255
14¼	841	950	1053	1156	1256	1354	1445	1536	1627	1714
14½	875	989	1096	1203	1308	1410	1505	1600	1695	1786
16¼	1134	1283	1423	1564	1701	1837	1963	2089	2214	2335
16½	1174	1328	1474	1620	1762	1902	2033	2164	2295	2420
18¼	1474	1669	1854	2039	2220	2398	2566	2733	2900	3061
18½	1520	1721	1912	2103	2290	2474	2647	2820	2993	3159
20¼	1862	2109	2346	2561	2812	3040	3255	3469	3683	3890
20½	1914	2168	2411	2654	2891	3125	3347	3568	3788	4001
22¼	2297	2604	2898	3190	3477	3761	4030	4297	4565	4823
22½	2355	2669	2971	3271	3565	3856	4133	4407	4682	4947
24¼	2780	3152	3510	3866	4215	4561	4890	5217	5545	5861
24½	2844	3224	3591	3955	4312	4667	5004	5338	5674	5998
26¼	3310	3754	4183	4609	5026	5441	5837	6228	6622	7002
26½	3380	3833	4271	4706	5133	5556	5961	6361	6764	7152
28¼	3888	4411	4916	5418	5911	6400	6869	7332	7798	8248
28½	3963	4497	5012	5524	6027	6526	7004	7476	7951	8411
30¼	4513	5121	5710	6295	6869	7439	7987	8527	9071	9597
30½	4594	5214	5813	6409	6994	7575	8133	8683	9237	9773
32¼	5185	5885	6564	7238	7900	8558	9190	9814	10443	11050
32½	5273	5985	6675	7361	8034	8703	9347	9982	10621	11239
34¼	5905	6704	7479	8248	9004	9756	10480	11193	11912	12608
34½	5999	6810	7598	8379	9147	9911	10647	11372	12103	12810
36¼	6672	7576	8454	9326	10181	11033	11855	12664	13480	14269
36½	6772	7689	8580	9465	10334	11198	12033	12854	13682	14484
38¼	7487	8503	9490	10470	11432	12390	13316	14227	15145	16035
38½	7593	8622	9624	10617	11594	12565	13505	14428	15360	16263
40¼	8349	9483	10586	11680	12756	13827	14863	15881	16909	17904
40½	8461	9609	10727	11836	12927	14012	15062	16094	17136	18145
42¼	9259	10517	11743	12958	14153	15342	16495	17627	18770	19877
42½	9376	10650	11892	13123	14333	15538	16705	17852	19010	20131
44¼	10216	11606	12960	14303	15623	16937	18213	19466	20730	21955
44½	10339	11746	13116	14476	15812	17143	18434	19702	20981	22222
46¼	11221	12748	14238	15714	17167	18612	20017	21396	22787	24136
46½	11350	12895	14402	15895	17365	18828	20249	21643	23051	24416
48¼	12273	13944	15576	17193	18783	20367	21907	23417	24943	26422
48½	12408	14098	15747	17382	18990	20593	22149	23677	25219	26715
50¼	13372	15195	16974	18738	20473	22201	23882	25531	27196	28811
50½	13513	15355	17153	18936	20689	22437	24135	25082	27485	29117
52¼	14519	16499	18433	20350	22236	24115	25944	27737	29548	31304
52½	14666	16666	18620	20556	22462	24360	26207	28019	29848	31623

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 33.—*Continued.*

Moments of Inertia of Four Angles with Unequal Legs, Axis X-X. Long Legs Turned Out.

Moments of Inertia of Four Angles, Axis X-X, Long Legs Turned Out.



For Distances Measured from Back to Back.

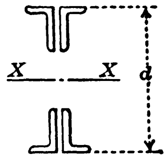
Size.	8″ × 6″, Long Legs Turned Out.									
Thick.	$\frac{7}{16}$″	$\frac{1}{2}$″	$\frac{9}{16}$″	$\frac{5}{8}$″	$\frac{11}{16}$″	$\frac{3}{4}$″	$\frac{13}{16}$″	$\frac{7}{8}$″	$\frac{15}{16}$″	1″
Area 4 ∟s	23.72	27.00	30.24	33.44	36.60	39.76	42.88	45.92	49.00	52.00
d″	Moments of Inertia About Axis X-X for Various Distances Back to Back of Angles, In.^{4}.									
54¼	15713	17858	19953	22029	24072	26108	28091	30034	31997	33902
54½	15866	18031	20147	22244	24307	26364	28365	30328	32310	34234
56¼	16955	19270	21533	23775	25982	28181	30323	32423	34545	36603
56½	17114	19450	21735	23998	26226	28446	30603	32728	34870	36948
58¼	18244	20736	23174	25588	27964	30333	32642	34904	37190	39409
58½	18409	20923	23383	25819	28217	30608	32938	35221	37528	39767
60¼	19581	22257	24875	27467	30020	32564	35046	37477	39934	42318
60½	19751	22450	25091	27707	30282	32850	35353	37805	40284	42689
62¼	20965	23831	26636	29414	32149	34876	37536	40142	42775	45331
62½	21141	24032	26860	29662	32420	35171	37853	40482	43137	45715
64¼	22396	25459	28458	31427	34351	37266	40112	42899	45715	48449
64½	22579	25667	28690	31684	34632	37572	40440	43250	46089	48846
66¼	23875	27142	30340	33508	36627	39737	42774	45747	48752	51670
66½	24064	27356	30580	33772	36916	40052	43112	46110	49139	52080
68¼	25402	28873	32283	35655	38975	42287	45521	48687	51888	54996
68½	25596	29099	32530	35928	39274	42612	45870	49061	52287	55419
70¼	26976	30669	34287	37869	41397	44916	48354	51719	55121	58425
70½	27176	30896	34541	38150	41705	45251	48714	52105	55532	58861
72¼	28597	32513	36351	40150	43892	47625	51273	54843	58453	61958
72½	28803	32747	36613	40440	44209	47970	51644	55240	58876	62407
74½	30478	34652	38745	42796	46787	50768	54659	58468	62318	66058
76½	32200	36611	40937	45219	49437	53646	57760	61787	65858	69812
78½	33969	38625	43190	47709	52161	56603	60947	65198	69495	73671
80½	35786	40692	45503	50266	54958	59640	64220	68700	73231	77633
82½	37651	42813	47877	52889	57828	62757	67578	72295	77065	81699
84½	39562	44988	50312	55800	60771	65953	71022	75981	80997	85870
86½	41522	47217	52806	58337	63788	69228	74552	79760	85026	90144
88½	43528	49500	55362	61162	66878	72583	78168	83630	89154	94523
90½	45583	51837	57977	64053	70041	76017	81869	87592	93380	99005
92½	47684	54228	60654	67011	73277	79531	85656	91646	97704	103591
94½	49833	56674	63390	70036	76586	83125	89529	95791	102125	108282
96½	52030	59173	66188	73128	79969	86798	93488	100029	106645	113076
98½	54274	61726	69045	76287	83425	90551	97532	104358	111263	117975
100½	56565	64333	71963	79512	86954	94383	101662	108779	115979	122977
102½	58904	66994	74942	82805	90556	98294	105878	113292	120792	128083
104½	61290	69709	77981	86164	94231	102285	110180	117897	125 04	133294
106½	63724	72478	81081	89590	97980	106356	114567	122594	130714	138608
108½	66205	75301	84241	93084	101802	110506	119041	127382	135822	144027
110½	68733	78178	87461	96644	105697	114736	123600	132263	141028	149549
112½	71309	81110	90742	100270	109665	119045	128244	137235	146331	155175

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 34.

Moments of Inertia of Four Angles with Unequal Legs, Axis X–X. Short Legs Turned Out.

Moments of Inertia of Four Angles, Axis X–X, Short Legs Turned Out.



For Distances Measured from Back to Back.

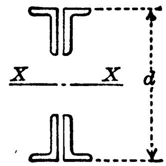
Size.	3″ × 2½″, Short Legs Out.					3½″ × 2½″, Short Legs Out.					4″ × 3″, Short Legs Out.				
Thick.	¼″	5/16″	⅜″	7/16″	½″	¼″	5/16″	⅜″	7/16″	½″	5/16″	⅜″	7/16″	½″	9/16″
Area 4 ∟s	5.24	6.48	7.68	8.88	10.00	5.76	7.12	8.44	9.72	11.00	8.36	9.92	11.48	13.00	14.48
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.^{4}.														
6½	33	41	47	53	59										
6¾	37	44	51	58	65										
7	40	48	56	64	71										
7¼	43	53	61	70	77										
7½	47	57	66	76	84	47	57	67	76	84					
7¾	51	62	72	82	91	51	62	72	82	92					
8	55	67	78	89	98	55	67	78	89	99					
8¼	59	72	84	95	106	60	72	84	96	107					
8½	63	77	90	102	114	64	78	91	103	115	88	103	118	131	144
8¾	68	83	96	110	122	69	83	97	111	124	95	111	127	141	155
9	72	88	103	118	131	73	89	104	119	133	101	119	136	151	166
9¼	77	94	110	125	140	78	95	112	127	142	108	127	145	161	178
9½	82	100	117	134	149	84	101	119	136	152	115	135	155	172	190
9¾	87	107	124	142	158	89	108	127	144	162	123	144	165	184	202
10	92	113	132	151	168	94	115	135	153	172	130	153	175	195	215
10¼	98	120	140	160	178	100	122	143	163	182	138	162	186	207	229
10½	104	127	148	169	189	106	129	151	173	193	147	172	197	220	242
10¾	109	134	156	179	200	112	136	160	183	205	155	182	209	233	257
11	115	141	165	189	211	118	144	169	193	216	164	192	221	246	272
11¼	121	149	174	199	222	125	152	179	204	228	173	203	233	260	287
11½	127	156	183	210	234	131	160	1[illegible]8	215	241	182	214	245	274	303
11¾	134	164	192	220	246	138	168	198	226	253	192	225	258	289	319
12	140	172	202	231	258	145	177	208	237	266	201	237	272	304	335
12¼	147	181	211	243	271	152	186	218	249	280	211	249	285	319	352
12½	154	189	222	254	284	159	195	229	261	293	222	261	299	335	370
12¾	161	198	232	266	297	167	204	240	274	308	232	273	314	351	388
13	168	207	242	278	311	175	213	251	287	322	243	286	329	368	406
13¼	176	216	253	290	325	182	223	262	300	337	254	299	344	385	425
13½	184	225	264	303	339	190	233	274	313	352	265	313	359	402	444
13¾	191	235	275	316	353	199	243	286	327	367	277	326	375	420	464
14	199	244	287	329	368	207	253	298	341	383	289	340	391	438	484
14¼	207	254	299	343	383	216	264	311	355	400	301	355	407	457	505
14½	215	266	310	357	399	224	275	323	370	415	313	369	424	476	526
14¾	223	275	323	371	415	233	286	336	385	432	326	384	442	495	548
15	232	285	335	385	431	242	297	349	400	450	339	400	459	515	570
15¼	241	296	348	400	447	252	308	363	415	467	352	415	477	535	592
15½	250	307	361	415	464	261	320	377	431	485	366	431	495	556	615
15¾	258	318	374	430	481	271	332	391	447	503	379	447	514	577	639
16	268	330	387	445	498	281	344	405	464	522	393	464	533	599	663
16¼	277	341	401	461	516	291	356	420	480	540	408	481	553	620	687
16½	287	353	415	477	534	301	369	434	497	560	422	498	573	643	712
16¾	297	365	429	493	552	311	381	450	515	579	437	515	593	665	737
18	348	428	503	579	648	366	449	529	606	682	514	607	699	785	870
18¼	358	441	519	596	669	377	463	546	625	704	531	626	721	810	898
18½	369	454	534	615	689	389	477	563	645	726	547	646	744	836	926
18¾	380	468	550	633	710	401	492	580	664	748	564	666	767	862	955

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 34.—*Continued.*

Moments of Inertia of Four Angles with Unequal Legs, Axis X–X. Short Legs Turned Out.

Moments of Inertia of Four Angles, Axis X–X, Short Legs Turned Out.



For Distances Measured from Back to Back.

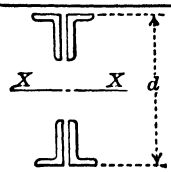
Size.	5″ × 3″, Short Legs Turned Out.													
Thick.	$\frac{5}{16}$″	$\frac{3}{8}$″	$\frac{7}{16}$″	$\frac{1}{2}$″	$\frac{9}{16}$″	$\frac{5}{8}$″	$\frac{11}{16}$″	Thick.	$\frac{3}{8}$″	$\frac{7}{16}$″	$\frac{1}{2}$″	$\frac{9}{16}$″	$\frac{5}{8}$″	$\frac{11}{16}$″
Area 4 Ls	9.60	11.44	13.24	15.00	16.72	18.44	20.12	Area 4 Ls	11.44	13.24	15.00	16.72	18.44	20.12
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.⁴.													
10½″	147	174	198	222	244	265	286	22¼	1046	1202	1356	1505	1649	1791
10¾	156	184	210	235	259	281	303	22½	1073	1234	1392	1544	1692	1838
11	165	195	222	249	274	298	322	24¼	1273	1464	1652	1835	2011	2186
11¼	174	206	235	263	290	315	340	24½	1303	1499	1692	1878	2059	2238
11½	184	217	248	278	307	333	360	26¼	1523	1753	1979	2198	2410	2620
11¾	194	229	261	293	323	352	380	26½	1556	1791	2022	2245	2463	2678
12	204	241	275	309	341	371	401	28¼	1796	2068	2335	2594	2847	3950
12¼	215	253	289	325	359	390	422	28½	1831	2109	2382	2646	2904	3158
12½	226	266	304	342	377	411	444	30¼	2091	2409	2721	3024	3320	3611
12¾	237	280	319	359	396	431	467	30½	2130	2454	2772	3080	3381	3678
13	248	293	335	376	416	453	490	32¼	2410	2777	3137	3487	3829	4166
13¼	260	307	351	394	436	475	514	32½	2451	2825	3192	3547	3896	4239
13½	272	321	367	413	456	497	538	34¼	2751	3172	3584	3984	4376	4762
13¾	284	336	384	432	477	520	563	34½	2796	3223	3642	4048	4447	4839
14	297	351	401	451	499	544	589	36¼	3116	3593	4060	4514	4960	5398
14¼	310	366	419	471	521	568	615	36½	3163	3647	4122	4583	5035	5480
14½	323	382	437	492	544	593	642	38¼	3503	4040	4566	5078	5580	6074
14¾	336	398	456	512	567	619	670	38½	3553	4098	4632	5151	5660	6162
15	350	414	475	533	591	645	698	40¼	3913	4514	5102	5675	6238	6791
15¼	364	431	494	556	615	671	727	40½	3966	4575	5172	5752	6322	6883
15½	379	448	514	578	640	698	757	42¼	4346	5014	5669	6305	6932	7547
15¾	393	467	534	601	665	726	787	42½	4402	5079	5742	6386	7021	7645
16	408	484	554	624	691	754	818	44¼	4802	5541	6265	6969	7663	8344
16¼	424	502	575	647	717	783	849	44½	4861	5609	6342	7055	7757	8447
16½	439	520	597	672	744	813	881	46¼	5281	6094	6891	7667	8431	9182
16¾	455	539	618	696	771	843	914	46½	5342	6165	6972	7756	8530	9289
17	472	558	641	721	799	873	947	48¼	5782	6674	7547	8398	9236	10059
17¼	488	578	663	747	827	904	981	48½	5847	6748	7632	8491	9339	10172
17½	505	598	686	773	856	936	1015	50¼	6307	7280	8234	9162	10078	10977
17¾	522	618	710	799	886	969	1051	50½	6374	7358	8322	92 0	10186	11094
18	539	639	733	826	916	1001	1086	52¼	6854	7913	8950	9960	10956	11935
18¼	557	660	758	854	946	1035	1123	52½	6924	7994	9042	10062	11069	12057
18½	575	682	782	882	977	1069	1160	54¼	7425	8572	9696	10791	11872	12933
18¾	593	703	808	910	1009	1104	1198	54½	7497	8657	9792	10897	11989	13060
20	690	818	939	1059	1174	1285	1395	56¼	8018	9258	10472	11655	12824	13971
20¼	710	841	967	1090	1209	1323	1437	56½	8094	9346	10572	11766	12946	14104
20½	730	866	995	1122	1244	1362	1479	58¼	8634	9970	11279	12553	13814	15050
20¾	751	890	1023	1154	1280	1401	1522	58½	8712	10061	11382	12668	13940	15187
21	772	915	1052	1186	1316	1441	1565	60¼	9273	10709	12115	13485	14840	16169
21¼	793	941	1081	1219	1353	1482	1609	60½	9354	10803	12222	13603	14971	16311
21½	815	966	1111	1253	1390	1523	1654	62¼	9935	11474	12981	14450	15903	17328
21¾	837	992	1141	1287	1428	1564	1699	62½	10019	11571	13092	14573	16038	17475

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 34.—*Continued.*

Moments of Inertia of Four Angles with Unequal Legs, Axis X–X. Short Legs Turned Out.

Moments of Inertia of Four Angles, Axis X-X, Short Legs Turned Out.



For Distances Measured from Back to Back.

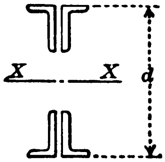
Size.	5″ × 3½″. Short Legs Turned Out.														
Thick.	⅜″	7/16″	½″	9/16″	⅝″	11/16″	¾″	Thick.	⅜″	7/16″	½″	9/16″	⅝″	11/16″	¾″
Area 4 ⌊s	12.20	14.12	16.00	17.88	19.68	21 48	23.24	Area 4 ⌊s	12.20	14.12	16.00	17.88	19.68	21.48	23.24
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.⁴														
10½	193	221	246	272	296	320	340	32¼	2601	3002	3388	3775	4143	4509	4859
10¾	204	234	261	288	314	339	361	32½	2646	3054	3446	3840	4214	4587	4942
11	216	247	276	305	332	359	382	34¼	2967	3426	3867	4309	4731	5149	5550
11¼	228	261	292	322	351	380	405	34½	3015	3481	3929	4379	4807	5233	5639
11½	240	275	308	340	371	401	428	36¼	3358	3877	4378	4880	5357	5833	6288
11¾	253	290	324	359	391	423	451	36½	3409	3936	4444	4953	5439	5921	6383
12	266	305	341	378	412	445	475	38¼	3773	4357	4920	5485	6024	6559	7072
12¼	280	321	359	398	433	469	501	38½	3827	4419	4990	5564	6110	6653	7173
12½	294	337	377	418	456	493	526	40¼	4213	4866	5495	6127	6729	7328	7903
12¾	308	354	396	439	478	517	553	40½	4270	4931	5569	6210	6820	7427	8010
13	323	370	415	460	502	543	580	42¼	4677	5402	6102	6805	7474	8140	8780
13¼	338	388	434	482	525	569	608	42½	4737	5471	6180	6892	7570	8245	8893
13½	353	406	454	504	550	595	637	44¼	5165	5967	6741	7518	8258	8995	9704
13¾	369	424	475	527	575	623	666	44½	5228	6039	6823	7610	8359	9105	9822
14	386	443	496	551	601	651	6 6	46¼	5678	6560	7412	8267	9082	9894	10674
14¼	402	462	518	575	627	679	727	46½	5744	6636	7498	8363	9188	10009	10798
14½	419	482	540	599	654	709	759	48¼	6215	7181	8115	9052	9945	10835	11691
14¾	437	502	563	625	682	739	791	48½	6285	7260	8205	9152	10055	10955	11821
15	454	522	586	650	710	770	824	50¼	6777	7830	8850	9872	10847	11819	12754
15¼	472	543	609	677	739	801	858	50½	6849	7913	8944	9977	10963	11945	12890
15½	491	564	633	704	768	833	892	52¼	7363	8508	9617	10728	11789	12846	13864
15¾	510	586	658	731	798	866	928	52½	7438	8594	9715	10838	11909	12977	14005
16	529	609	683	759	829	899	964	54¼	7973	9214	10415	11620	12770	13915	15020
16¼	549	631	709	788	860	933	1000	54½	8052	9304	10518	11734	12895	14052	15167
16½	569	654	735	817	892	968	1038	56¼	8608	9948	11246	12548	13790	15028	16223
16¾	589	678	761	846	925	1 03	1076	56½	8689	10041	11352	12667	13921	15170	16376
18	697	803	902	1003	1097	1190	1277	58¼	9267	10710	12109	13512	14850	16184	17472
18¼	720	829	932	1036	1133	1230	1320	58½	9352	10807	12219	13635	14985	16332	17631
18½	743	856	962	1070	1170	1270	1363	60¼	9950	11501	13004	14511	15949	17383	18768
18¾	767	883	992	1104	1207	1311	1407	60½	10038	11601	13118	14639	16089	17536	18932
20¼	915	1055	1186	1319	1445	1569	1686	62¼	10658	12319	13931	15546	17088	18625	20110
20½	942	1085	1221	1357	1487	1615	1735	62½	10749	12424	14049	15678	17233	18783	20280
22¼	1135	1309	1473	1639	1796	1952	2099	64¼	11391	13166	14890	16617	18266	19909	21498
22½	1165	1342	1511	1682	1843	2003	2153	64½	11485	13274	15012	16753	18416	20073	21675
24¼	1379	1591	1792	1995	2187	2377	2558	66¼	12148	14042	15881	17724	19483	21237	22934
24½	1412	1628	1834	2042	2239	2434	2618	66½	12245	14153	16007	17864	19638	21406	23116
26¼	1648	1901	2143	2386	2617	2846	3063	68¼	12929	14945	16904	18866	20739	22608	24415
26½	1684	1942	2189	2438	2674	2908	3129	68½	13029	15060	17034	19011	20899	22782	24603
28¼	1941	2240	2526	2813	3086	3357	3615	70¼	13734	15877	17958	20044	22035	24021	25943
28½	1980	2284	2576	2869	3148	3424	3687	70½	13837	15996	18093	20194	22200	24201	26137
30¼	2259	2607	2941	3276	3595	3912	4214	72¼	14564	16837	19045	21258	23371	25478	27518
30½	2301	2655	2995	3337	3661	3984	4291	72½	14670	16959	19183	21412	23540	25663	27717

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 34.—*Continued.*

Moments of Inertia of Four Angles with Unequal Legs, Axis X–X. Short Legs Turned Out.

Moments of Inertia
of Four Angles,
Axis X–X,
Short Legs Turned Out.



For Distances
Measured
from
Back to Back.

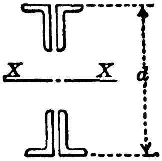
Size.	6″ × 4″, Short Legs Turned Out.										
Thick.	⅜″	7/16″	½″	9/16″	⅝″	11/16″	¾″	13/16″	⅞″	15/16″	1″
Area 4 ∟s	14.44	16.72	19.00	21.24	23.44	25.60	27.76	29.88	31.92	34.00	36.00
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.⁴.										
12½″	322	370	414	459	502	541	581	619	655	691	722
14¼	442	508	571	633	693	748	805	858	911	962	1007
14½	461	530	595	660	723	781	840	897	951	1005	1052
16¼	606	697	785	871	955	1033	1112	1188	1262	1335	1400
16½	629	723	814	904	991	1072	1155	1235	1311	1386	1454
18¼	799	920	1037	1152	1264	1369	1476	1578	1677	1776	1864
18½	825	950	1071	1190	1306	1415	1525	1632	1734	1836	1928
20¼	1021	1177	1327	1476	1620	1756	1895	2028	2156	2285	2401
20½	1051	1211	1366	151	1668	1808	1951	2089	2221	2353	2473
22¼	1272	1466	1655	1842	2023	2195	2369	2537	2699	2862	3010
22½	1305	1505	1699	1890	2077	2253	2432	2606	2772	2939	3091
24¼	1552	1790	2021	2250	2473	2685	2899	3107	3306	3507	3691
24½	1589	1832	2070	2304	2533	2749	2969	3183	3387	3592	3781
26¼	1860	2146	2425	2701	2970	3226	3485	3736	3977	4220	4443
26½	1901	2193	2479	2760	3035	3297	3562	3819	4066	4314	4543
28¼	2198	2536	2868	3195	3513	3818	4126	4424	4711	5001	5268
28½	2242	2587	2925	3259	3585	3895	4210	4516	4808	5103	5376
30¼	2564	2960	3348	3730	4104	4461	4822	5173	5510	5850	6165
30½	2612	3015	3410	3800	4181	4545	4913	5272	5614	5961	6282
32¼	2959	3417	3866	4309	4741	5156	5574	5981	6372	6767	7134
32½	3011	3476	3933	4384	4824	5246	5672	6087	6484	6886	7260
34¼	3383	3907	4422	4930	5425	5901	6382	6849	7298	7752	8174
34½	3439	3971	4494	5010	5514	5998	6486	6963	7418	7880	8310
36¼	3836	4431	5016	5593	6156	6698	7245	7777	8288	8805	9287
36½	3895	4499	5093	5679	6251	6801	7356	7898	8416	8941	9431
38¼	4318	4988	5648	6299	6934	7546	8163	8764	9341	9926	10472
38½	4381	5060	5730	6390	7035	7656	8282	8893	9478	10071	10625
40¼	4829	5579	6318	7047	7759	8446	9137	9812	10459	11115	11729
40½	4895	5655	6405	7143	7866	8562	9263	9948	10603	11268	11891
42¼	5369	6203	7026	7838	8631	9396	10167	10919	11640	12372	13058
42½	5438	6283	7118	7940	8743	9519	10300	11062	11793	12534	13229
44¼	5937	6861	7773	8671	9550	10398	11252	12085	12885	13697	14458
44½	6010	6945	7868	8778	9668	10527	11392	12237	13046	13867	14638
46¼	6535	7552	8557	9547	10515	11451	12393	13312	14194	15090	15931
46½	6611	7640	8657	9659	10639	11586	12539	13471	14363	15269	16120
48¼	7161	8276	9379	10465	11527	12555	13589	14598	15567	16551	17476
48½	7241	8369	9484	10583	11657	12697	13742	14764	15744	16738	17674
50¼	7816	9034	10239	11426	12587	13710	14841	15944	17004	18080	19093
50½	7900	9131	10349	11549	12722	13858	15001	16118	17189	18275	19300
52¼	8500	9826	11137	12429	13693	14917	16148	17350	18505	19677	20781
52½	8588	9927	11252	12557	13834	15071	16315	17531	18697	19881	20997

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 34.—*Continued.*

Moments of Inertia of Four Angles with Unequal Legs, Axis X–X. Short Legs Turned Out.

Moments of Inertia of Four Angles, Axis X-X, Short Legs Turned Out.



For Distances Measured from Back to Back.

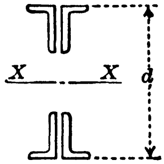
Size.	6″ × 4″, Short Legs Turned Out.										
Thick.	$\frac{3}{8}$″	$\frac{7}{16}$″	$\frac{1}{2}$″	$\frac{9}{16}$″	$\frac{5}{8}$″	$\frac{11}{16}$″	$\frac{3}{4}$″	$\frac{13}{16}$″	$\frac{7}{8}$″	$\frac{15}{16}$″	1″
Area 4 ∠s	14.44	16.72	19.00	21.24	23.44	25.60	27.76	29.88	31.92	34.00	36.00
d″	Moments of Inertia About Axis X–X for Various Distances Back to Back of Angles, In.⁴										
54¼″	9213	10650	12073	13475	14846	16175	17511	18816	20069	21342	22542
54½	9304	10756	12193	13608	14993	16335	17685	19004	20269	21554	22767
56¼	9955	11509	13047	14563	16046	17484	18929	20341	21697	23075	24375
56½	10049	11618	13172	14701	16199	17651	19110	20537	21906	23296	24609
58¼	10725	12400	14059	15693	17292	18844	20403	21926	23389	24876	26280
58½	10824	12514	14189	15837	17452	19016	20591	22130	23606	25105	26523
60¼	11525	13325	15110	16866	18586	20255	21932	23571	25145	26744	28256
60½	11627	13443	15244	17016	18751	20434	22127	23782	25370	26983	28509
62¼	12353	14284	16198	18082	19927	21718	23517	25276	26965	28681	30305
62½	12459	14406	16336	18237	20097	21903	23719	25494	27197	28928	30566
64¼	13211	15276	17324	19340	21314	23231	25157	27040	28849	30686	32426
64½	13320	15402	17467	19500	21491	23423	25366	27266	29089	30942	32696
66¼	14097	16301	18488	20641	22748	24796	26853	28065	30796	32759	34619
66½	14210	16432	18636	20806	22931	24994	27069	29098	31045	33023	34898
68¼	15012	17360	19690	21984	24229	26412	28604	30748	32807	34900	36883
68½	15128	17495	19843	22154	24418	26617	28827	30989	33064	35173	37172
70¼	15956	18453	20930	23369	25758	28080	30411	32692	34882	37109	39220
70½	16076	18591	21088	23545	25952	28291	30641	32940	35147	37390	39517
72¼	16929	19578	22208	24797	27332	29798	32274	34696	37021	39386	41629
72½	17052	19721	22371	24978	27533	30016	32510	34951	37294	39676	41935
74½	18058	20885	23692	26454	29160	31792	34435	37022	39505	42029	44425
76½	19092	22082	25051	27972	30835	33619	36416	39152	41780	44451	46987
78½	20155	23312	26447	29533	32556	35498	38452	41343	44118	46940	49620
80½	21247	24576	27882	31136	34325	37427	40543	43593	46520	49498	52326
82½	22368	25873	29355	32782	36140	39408	42690	45902	48986	52123	55104
84½	23517	27203	30866	34470	38002	41440	44892	48272	51516	54817	57954
86½	24696	28567	32415	36201	39911	43524	47150	50701	54110	57578	60875
88½	25903	29965	34002	37974	41867	45658	49464	53190	56768	60408	63869
90½	27140	31396	35627	39789	43869	47844	51833	55739	59489	63305	66935
92½	28405	32860	37290	41647	45919	50081	54258	58347	62275	66271	70073
94½	29699	34358	38990	43548	48015	52369	56738	61016	65124	69304	73282
96½	31022	35889	40729	45491	50159	54708	59273	63744	68037	72406	76564
98½	32374	37454	42506	47476	52349	57099	61864	66531	71014	75575	79918
100½	33755	39052	44321	49504	54586	59541	64511	69379	74054	78812	83344
102½	35164	40683	46174	51575	56870	62034	67213	72286	77159	82118	86841
104½	36603	42348	48065	53688	59201	64578	69971	75253	80327	85491	90411
106½	38070	44047	49994	55843	61579	67173	72784	78280	83560	88933	94053
108½	39566	45779	51961	58041	64003	69820	75653	81367	86856	92442	97767
110½	41092	47544	53966	60282	66475	72517	78577	84513	90216	96020	101553
112½	42646	49343	56008	62564	68993	75267	81557	87719	93639	99665	105410

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 34.—*Continued.*

Moments of Inertia of Four Angles with Unequal Legs, Axis X–X. Short Legs Turned Out.

Moments of Inertia of Four Angles, Axis X–X, Short Legs Turned Out.



For Distances Measured from Back to Back.

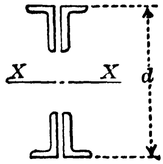
Size.	8″ × 6″, Short Legs Turned Out.									
Thick.	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
Area 4 ∟s	23.72	27.00	30.24	33.44	36.60	39.76	42.88	45.92	49.00	52.00
d″	Moments of Inertia About Axis X-X for Various Distances Back to Back of Angles, In.⁴.									
16½″	955	1079	1197	1314	1429	1541	1645	1750	1854	1954
18¼	1214	1373	1524	1675	1822	1967	2103	2238	2373	2503
18½	1254	1418	1575	1731	1883	2033	2174	2314	2454	2588
20¼	1554	1759	1955	2150	2341	2529	2706	2883	3059	3229
20½	1600	1812	2013	2215	2411	2605	2788	2970	3152	3327
22¼	1942	2200	2447	2692	2933	3170	3395	3619	3842	4058
22½	1994	2259	2512	2765	3012	3256	3488	3717	3947	4169
24¼	2377	2694	2999	3301	3598	3891	4170	4447	4724	4991
24½	2435	2760	3072	3382	3686	3987	4273	4557	4841	5115
26¼	2860	3243	3611	3977	4336	4692	5031	53[illegible]6	5703	6029
26½	2924	3315	3692	4066	4433	4797	5144	5488	5833	6166
28¼	3390	3845	4284	4720	5147	5572	5977	6378	6781	7170
28½	3460	3924	4372	4818	5254	5687	6101	6511	6923	7320
30¼	3968	4501	5017	5530	6032	6531	7009	7482	7956	8416
30½	4043	4587	5113	5635	6148	6656	7144	7626	8110	8579
32¼	4593	5212	5811	6406	6990	7570	8127	8677	9230	9765
32½	4674	5304	5914	6520	7115	7705	8273	8833	9396	9941
34¼	5265	5976	6665	7349	8021	8688	9331	9964	10602	11218
34½	5353	6075	6776	7472	8155	8834	9487	10131	10780	11407
36¼	5985	6794	7580	8360	9125	9886	10620	11343	12071	12776
36½	6078	6900	7698	8491	9268	10042	10787	11522	12262	12978
38¼	6752	7667	8555	9437	10303	11164	11995	12814	13639	14437
38½	6852	7780	8681	9576	10455	11329	12173	13004	13841	14652
40¼	7567	8593	9591	10581	11553	12521	13456	14376	15304	16203
40½	7672	8713	9725	10728	11715	12696	13645	14578	15519	16431
42¼	8429	9573	10687	11791	12877	13957	15003	16031	17068	18072
42½	8540	9700	10828	11948	13048	14143	15202	16244	17295	18313
44¼	9339	10608	11844	13069	14274	15473	16635	17777	18929	20045
44½	9456	10741	11993	13234	14454	15668	16845	18002	19169	20299
46¼	10296	11696	13061	14414	15744	17069	18354	19615	20889	22123
46½	10419	11836	13217	14587	15933	17274	18574	19852	21140	22390
48¼	11301	12839	14339	15825	17288	18744	20158	21545	22946	24304
48½	11430	12985	14502	16007	17486	18959	20389	21793	23210	24584
50¼	12353	14035	15677	17304	18904	20499	22047	23567	25102	26590
50½	12487	14188	15848	17493	19111	20734	22290	23827	25378	26883
52¼	13452	15285	17075	18849	20504	22333	24023	25681	27355	28979
52½	135[illegible]3	15445	17254	19047	20810	22568	24277	25952	27644	29285
54¼	14599	16590	18534	20461	22357	24246	26084	27887	29707	31472
54½	14746	16757	18721	20667	22583	24491	26349	28169	30007	31791
56¼	15793	17948	20054	22140	24193	26240	28231	30184	32156	34070
56½	15946	18122	20248	22355	24428	26494	28506	30478	32469	34402

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 34.—*Continued.*

Moments of Inertia of Four Angles with Unequal Legs, Axis X-X. Short Legs Turned Out.

Moments of Inertia of Four Angles, Axis X-X, Short Legs Turned Out.



For Distances Measured from Back to Back.

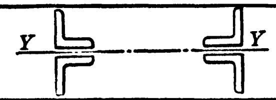
Size.	8″ × 6″, Short Legs Out.									
Thick.	$\frac{7}{16}$″	$\frac{1}{2}$″	$\frac{9}{16}$″	$\frac{5}{8}$″	$\frac{11}{16}$″	$\frac{3}{4}$″	$\frac{13}{16}$″	$\frac{7}{8}$″	$\frac{15}{16}$″	1″
Area 4 ⌊s	23.72	27.00	30.24	33.44	36.60	39.76	42.88	45.92	49.00	52.00
d″	Moments of Inertia About Axis X-X for Various Distances Back to Back of Angles, In.^{4}									
$58\frac{1}{4}$″	17035	19360	21634	23886	26103	28312	30464	32573	34704	36771
$58\frac{1}{2}$	17194	19541	21836	24109	26347	28577	30750	32878	35029	37116
$60\frac{1}{4}$	18324	20827	23274	25699	28085	30465	32782	35054	37349	39577
$60\frac{1}{2}$	18489	21014	23484	25930	28338	30739	33079	35371	37687	39935
$62\frac{1}{4}$	19661	22347	24975	27578	30141	32696	35187	37627	40093	42486
$62\frac{1}{2}$	19831	22541	25192	27818	30403	32981	35494	37955	40442	42857
$64\frac{1}{4}$	21045	23922	26737	29525	32270	35007	37677	40292	42934	45499
$64\frac{1}{2}$	21221	24122	26961	29773	32541	35302	37995	40631	43296	45883
$66\frac{1}{4}$	22476	25550	28559	31538	34472	37398	40252	43048	45874	48617
$66\frac{1}{2}$	22659	25757	28791	31795	34753	37703	40581	43400	46248	49014
$68\frac{1}{4}$	23955	27232	30441	33619	36748	39869	42914	45897	48911	51838
$68\frac{1}{2}$	24143	27446	30681	33884	37037	40183	43254	46259	49298	52248
$70\frac{1}{4}$	25482	28969	32384	35766	39096	42418	45661	48837	52047	55164
$70\frac{1}{2}$	25676	29190	32631	36039	39395	42743	46012	49211	52446	55587
$72\frac{1}{4}$	27056	30759	34388	37980	41518	45048	48494	51869	55280	58593
$72\frac{1}{2}$	27256	30987	34642	38261	41826	45382	48856	52255	55691	59029
$74\frac{1}{2}$	28883	32838	36714	40551	44330	48101	51785	55390	59035	62575
$76\frac{1}{2}$	30557	34743	38846	42907	46908	50899	54800	58617	62477	66226
$78\frac{1}{2}$	32279	36702	41038	45330	49558	53777	57901	61937	66017	69980
$80\frac{1}{2}$	34049	38715	43291	47820	52282	56734	61088	65347	69654	73839
$82\frac{1}{2}$	35866	40782	45604	50377	55079	59771	64361	68850	73390	77801
$84\frac{1}{2}$	37730	42903	47978	53000	57949	62887	67719	72445	77224	81867
$86\frac{1}{2}$	39642	45078	50412	55691	60893	66083	71163	76131	81156	86038
$88\frac{1}{2}$	41601	47308	52907	58449	63909	69359	74693	79910	85185	90312
$90\frac{1}{2}$	43608	49591	55463	61273	66999	72714	78309	83780	89313	94691
$92\frac{1}{2}$	45662	51928	58078	64164	70162	76148	82010	87742	93539	99173
$94\frac{1}{2}$	47764	54319	60755	67122	73398	79662	85797	91796	97863	103759
$96\frac{1}{2}$	49913	56764	63491	70147	76707	83256	89670	95941	102284	108450
$98\frac{1}{2}$	52109	59263	66288	73239	80090	86929	93629	100179	106804	113244
$100\frac{1}{2}$	54353	61816	69146	76398	83546	90681	97674	104508	114422	118143
$102\frac{1}{2}$	56645	64423	72064	79623	87075	94513	101804	108929	116138	123145
$104\frac{1}{2}$	58983	67085	75043	82916	90677	98425	106020	113442	120951	128251
$106\frac{1}{2}$	61370	69800	78082	86275	94352	102416	110321	118047	125863	133462
$108\frac{1}{2}$	63803	72569	81182	89702	98101	106487	114709	122744	130873	138776
$110\frac{1}{2}$	66284	75392	84342	93195	101923	110637	119182	127532	135981	144195
$112\frac{1}{2}$	68813	78269	87562	96755	105818	114867	123741	132113	141186	149717
$114\frac{1}{2}$	71389	81200	90843	100382	109786	119176	128386	137385	146490	155343
$116\frac{1}{2}$	74012	84185	94185	104075	113827	123564	133116	142449	151892	161074
$118\frac{1}{2}$	76683	87224	97587	107836	117942	128033	137993	147605	157392	166908
$120\frac{1}{2}$	79402	90318	101049	111664	122129	132580	142835	152853	162990	172847

Moment of Inertia of Net Area = Tabular Value × Net Area ÷ Gross Area (approx.).

TABLE 35.

Moments of Inertia of Four Angles with Equal Legs, Axis Y–Y.

Moments of Inertia of Four Angles, Axis Y-Y, Equal Legs.



For Distances Measured from Back to Back.

Size of Angles.	Area, Four Angles.	Distance Back to Back in Inches.							Size of Angles.	Area, Four Angles.	Distance Back to Back in Inches.						
In.	In.²	0	¼	5/16	⅜	½	⅝	¾	In.	In.²	0	¼	5/16	⅜	½	⅝	¾
2x2x3/16	2.84	2.1	2.5	2.6	2.8	3.1	3.4	3.7	2½x2½x¼	4.76	5.3	6.2	6.5	6.7	7.3	7.9	8.5
" ¼	3.76	2.7	3.3	3.5	3.7	4.1	4.5	4.9	" 5/16	5.88	6.6	7.8	8.1	8.5	9.2	9.9	10.7
" 5/16	4.60	3.4	4.2	4.4	4.6	5.1	5.6	6.1	" ⅜	6.92	7.9	9.3	9.7	10.1	11.0	11.9	12.8
" ⅜	5.44	4.2	5.1	5.3	5.5	6.2	6.7	7.3	" 7/16	8.00	9.3	11.0	11.5	11.9	12.9	14.0	15.1
3x3x¼	5.76	9.0	10.3	10.7	11.0	11.8	12.6	13.5	3½x3½x¼	6.76	14.2	16.1	16.6	17.1	18.1	19.2	20.3
" 5/16	7.12	11.4	13.1	13.5	14.0	15.0	16.0	17.1	" 5/16	8.36	18.0	20.2	20.8	21.4	22.7	24.0	25.4
" ⅜	8.44	13.7	15.7	16.3	16.8	18.0	19.2	20.6	" ⅜	9.92	21.8	24.3	25.0	25.7	27.2	28.8	30.5
" 7/16	9.72	16.0	18.4	19.0	19.7	21.0	22.5	24.0	" 7/16	11.48	25.4	28.6	29.5	30.3	32.1	34.0	36.0
" ½	11.00	18.4	21.1	21.9	22.6	24.2	25.9	27.6	" ½	13.00	29.2	32.8	33.7	34.7	36.8	39.0	41.3
" 9/16	12.24	20.8	23.8	24.7	25.6	27.4	29.2	31.2	" 9/16	14.48	32.8	37.0	38.1	39.2	41.6	44.1	46.7
" ⅝	13.44	23.3	26.5	27.5	28.5	30.5	32.5	35.1	" ⅝	15.92	36.5	41.2	42.5	43.7	46.3	49.1	52.0

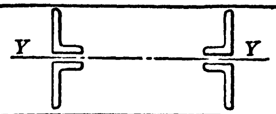
Size of Angles.	Area, Four Angles.	Distance Back to Back of Angles in Inches.												
In.	In.²	0	¼	5/16	⅜	7/16	½	9/16	⅝	¾	⅞	1	1⅛	1¼
4x4x¼	7.76	21.5	23.6	24.3	25.0	25.6	26.3	26.9	27.4	28.9	....	....	....	....
" 5/16	9.60	26.9	29.7	30.5	31.3	32.1	32.9	33.7	34.5	36.3	....	....	....	....
" ⅜	11.44	32.3	35.8	36.7	37.6	38.6	39.5	40.5	41.6	43.7	....	....	....	....
" 7/16	13.24	37.7	41.7	42.8	43.9	45.1	46.2	47.4	48.6	51.1	....	....	....	....
" ½	15.00	43.1	47.8	49.0	50.3	51.6	52.9	54.3	55.7	58.5	....	....	....	....
" 9/16	16.72	49.0	54.3	55.7	57.1	58.6	60.1	61.6	63.2	66.5	....	....	....	....
" ⅝	18.44	54.5	60.5	62.1	63.7	65.3	67.0	68.7	70.5	74.1	....	....	....	....
5x5x⅜	14.44	62.7	68.1	69.5	70.9	72.3	73.8	75.3	76.8	79.9	....	....	....	....
" 7/16	16.72	73.2	79.5	81.1	82.7	84.4	86.1	87.9	89.7	93.3	....	....	....	....
" ½	19.00	84.0	90.9	92.8	94.7	96.7	98.6	100.6	102.7	106.9	....	....	....	....
" 9/16	21.24	94.8	103.1	105.2	107.4	109.6	111.9	114.2	116.5	121.3	....	....	....	....
" ⅝	23.44	105.6	114.7	117.1	119.5	122.0	124.5	127.0	129.6	135.0	....	....	....	....
" 11/16	25.60	116.4	126.3	129.0	131.6	134.4	137.1	140.0	142.8	148.7	....	....	....	....
" ¾	27.76	126.8	138.1	141.0	143.9	146.9	150.0	153.0	156.2	162.6	....	....	....	....
6x6x⅜	17.44	108.5	....	....	119.8	121.8	123.9	125.9	128.1	132.4	136.8	141.4	146.2	151.0
" 7/16	20.24	126.5	....	....	139.8	142.2	144.6	147.0	149.5	154.5	159.8	165.2	170.7	176.5
" ½	23.00	144.6	....	....	159.8	162.5	165.3	168.1	171.0	176.8	182.8	188.9	195.3	201.8
" 9/16	25.72	163.5	....	....	180.9	184.0	187.1	190.3	193.5	200.1	206.9	213.9	221.1	228.5
" ⅝	28.44	181.8	....	....	201.2	204.6	208.1	211.7	215.3	222.7	230.3	238.1	246.1	254.4
" 11/16	31.12	200.1	....	....	221.6	225.4	229.2	233.2	237.1	245.3	253.7	262.3	266.7	275.7
" ¾	33.76	219.6	....	....	243.3	247.5	251.7	256.0	260.4	269.4	278.6	288.1	297.9	307.9
" ⅞	38.92	256.6	....	....	284.6	289.5	294.4	299.5	304.8	315.2	326.1	337.2	348.7	360.3
" 1	44.00	294.0	....	....	326.3	332.0	337.7	343.5	349.5	361.6	374.1	386.9	400.0	413.5
8x8x½	31.00	343.2	....	....	369.8	374.4	379.1	383.8	388.7	398.5	408.5	418.9	429.4	440.2
" 9/16	34.72	385.9	....	....	415.9	421.2	426.5	431.8	437.3	448.4	459.7	471.3	483.2	495.4
" ⅝	38.44	428.8	....	....	462.4	468.2	474.1	480.1	486.2	498.5	511.2	524.2	537.4	551.0
" 11/16	42.12	471.8	....	....	508.8	515.3	521.8	528.4	535.1	548.8	562.7	577.0	591.7	606.6
" ¾	45.76	516.8	....	....	557.6	564.7	571.9	579.2	586.5	601.6	616.9	632.6	648.7	665.1
" ⅞	52.92	603.2	....	....	651.1	659.4	667.9	676.4	685.1	702.7	720.8	739.2	758.1	777.3
" 1	60.00	692.9	....	....	748.4	758.0	767.8	777.7	787.7	808.0	828.8	850.1	871.8	894.1
" 1⅛	66.92	780.8	....	....	843.4	854.3	865.4	876.6	887.9	910.9	934.5	958.5	983.1	1008.3

Radii of Gyration about Axis Y–Y, same as given in table of Radii of Gyration of Two Angles.

TABLE 36.

Moments of Inertia of Four Angles with Unequal Legs, Axis Y-Y. Long Legs Out.

Moments of Inertia of Four Angles, Axis Y-Y, Long Legs Turned Out.



For Distances Measured from Back to Back.

Size of Angles. In	Area, Four Angles. In.²	0	¼	5/16	⅜	½	⅝	¾
2½x2x3/16	3.24	3.9	4.6	4.8	5.0	5.4	5.8	6.2
" ¼	4.24	5.2	6.2	6.4	6.7	7.2	7.8	8.4
" 5/16	5.24	6.6	7.7	8.1	8.4	9.1	9.8	10.5
" ⅜	6.20	7.9	9.3	9.7	10.1	10.9	11.7	12.6
" 7/16	7.12	9.3	10.9	11.3	11.8	12.7	13.7	14.7
3½x2½x¼	5.76	14.3	16.0	16.4	16.9	17.9	18.9	19.9
" 5/16	7.12	18.1	20.2	20.7	21.3	22.5	23.8	25.1
" ⅜	8.44	21.4	24.2	24.9	25.6	27.0	28.5	30.1
" 7/16	9.72	25.1	28.2	29.0	29.8	31.5	33.3	35.1
" ½	11.00	28.6	32.3	33.2	34.1	36.1	38.1	40.2
4x3x¼	6.76	21.3	23.7	24.3	24.8	26.1	27.4	28.8
" 5/16	8.36	26.8	29.6	30.3	31.0	32.6	34.2	35.9
" ⅜	9.92	32.1	35.4	36.3	37.2	39.1	41.0	43.0
" 7/16	11.48	37.5	41.4	42 4	43.5	45.7	47.9	50.3
" ½	13.00	43.2	47.7	48.9	50.1	52.7	55.3	58.0
" 9/16	14.48	48.6	53.7	55.1	56.4	59.3	62.2	65.3
" ⅝	15.92	54.0	59.9	61.3	62.7	65.9	69.0	72.6

Size of Angles. In.	Area, Four Angles. In.²	0	¼	5/16	⅜	½	⅝	¾
3x2½x¼	5.24	9.0	10.3	10.6	11.0	11.7	12.5	13.3
" 5/16	6.48	11.2	12.9	13.3	13.8	14.7	15.7	16.7
" ⅜	7.68	13.8	15.7	16.2	16.8	17.9	19.1	20.3
" 7/16	8.88	16.0	18.4	19.0	19.6	21.0	22.4	23.8
" ½	10.00	18.3	21.0	21.7	22.4	24.0	25.6	27.2
3½x3x¼	6.24	14.4	16.1	16.6	17.0	18.0	19.0	20.1
" 5/16	7.72	18.0	20.2	20.7	21.3	22.6	23.9	25.2
" ⅜	9.20	21.6	24.3	25.0	25.7	27.2	28.8	30.4
" 7/16	10.60	25.2	28.3	29.1	30.0	31.7	33.5	35.4
" ½	12.00	29.2	32.7	33.7	34.6	36.7	38.8	41.0
5x3x5/16	9.60	52.3	56.3	57.4	58.5	60.8	63.2	65.6
" ⅜	11.44	62.7	67.6	68.9	70.2	73.0	75.8	78.7
" 7/16	13.24	73.2	79.3	80.8	82.4	85.6	89.0	92.4
" ½	15.00	84.0	90.5	92.3	94.1	97.8	101.6	105.5
" 9/16	16.72	94.0	101.8	103.8	105.8	110.0	114.2	118.7
" ⅝	18.44	105.3	113.8	116.1	118.3	123.0	127.8	132.7
" 11/16	20.12	115.9	125.2	127.7	130.2	135.3	140.6	146.1

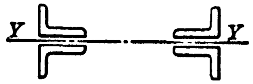
Size of Angles. In.	Area, Four Angles. In.²	Distance Back to Back of Angles in Inches. 0	¼	5/16	⅜	7/16	½	9/16	⅝	¾	⅞	1	1¼	1½
5x3½x5/16	10.24	52.3	56.5	57.6	58.8	59.9	61.1	62.3	63.5	65.9	....	....	....	....
" ⅜	12.20	62.7	67.8	69.2	70.5	71.9	73.3	74.7	76.2	79.2	....	....	....	....
" 7/16	14.12	73.1	79.1	80.7	82.2	83.9	85.5	87.2	88.9	92.4	....	....	....	....
" ½	16.00	84.0	90.9	92.7	94.6	96.4	98.3	100.2	102.2	106.2	....	....	....	....
" 9/16	17.88	94.6	102.4	104.4	106.5	108.6	110.7	112.9	115.1	119.6	....	....	....	....
" ⅝	19.68	105.0	113.7	115.9	118.2	120.6	123.0	125.4	127.8	132.9	....	....	....	....
" 11/16	21.48	115.6	125.1	127.6	130.1	132.7	135.3	138.0	140.7	146.2	....	....	....	....
" ¾	23.24	126.8	137.4	140.1	142.9	145.8	148.7	151.6	154.6	160.6	....	....	....	....
6x4x⅜	14.44	108.2	115.5	117.3	119.2	121.2	123.1	125.1	127.1	131.3	....	....	....	....
" 7/16	16.72	126.1	134.5	136.7	139.0	141.2	143.5	145.8	148.2	153.0	....	....	....	....
" ½	19.00	144.8	154.6	157.1	159.7	162.3	164.9	167.6	170.3	175.9	....	....	....	....
" 9/16	21.24	162.9	173.9	176.7	179.6	182.6	185.5	188.5	191.6	197.9	....	....	....	....
" ⅝	23.44	180.9	193.1	196.3	199.5	202.8	206.1	209.5	212.9	219.9	....	....	....	....
" 11/16	25.60	200.1	213.7	217.2	220.8	224.4	228.1	231.8	235.6	243.3	....	....	....	....
" ¾	27.76	218.1	233.0	236.9	240.8	244.7	248.8	252.8	256.9	265.4	....	....	....	....
" ⅞	31.92	254.2	271.8	276.3	280.9	285.5	290.2	295.0	299.8	309.6	....	....	....	....
" 1	36.00	292.8	312.6	317.8	323.1	328.4	333.8	339.3	344.9	356.2	....	....	....	....
8x6x7/16	23.72	299.2	....	....	321.9	325.8	329.8	333.9	337.9	346.2	354.7	363.3	372.1	381.2
" ½	27.00	342.0	....	....	367.9	372.4	377.0	381.6	386.2	395.7	405.4	415.3	425.4	435.8
" 9/16	30.24	386.2	....	....	415.5	420.6	425.7	431.0	436.3	447.0	458.0	469.2	480.7	497.4
" ⅝	33.44	428.8	....	....	461.5	467.2	473.0	478.8	484.7	496.7	508.9	521.4	534.2	547.2
" 11/16	36.60	471.2	....	....	507.5	513.8	520.2	526.6	533.1	546.3	559.8	573.5	587.5	601.9
" ¾	39.76	514.0	....	....	553.8	560.7	567.6	574.7	581.8	596.2	611.0	626.0	641.4	657.1
" ⅞	45.92	602.0	....	....	648.6	656.7	664.9	673.1	681.5	698.5	715.8	733.5	751.5	769.9
" 1	52.00	688.0	....	....	741.8	751.1	760.5	769.9	779.5	799.0	818.8	839.1	859.8	880.9

Radii of Gyration about Axis *Y–Y*, same as given in table of Radii of Gyration of Two Angles.

TABLE 37.

Moments of Inertia of Four Angles with Unequal Legs, Axis Y–Y. Short Legs Out.

Moments of Inertia of Four Angles, Axis Y–Y, Short Legs Turned Out.



For Distances Measured from Back to Back.

Size of Angles. In.	Area, Four Angles. In.²	Distance Back to Back in Inches. 0	¼	5/16	⅜	½	⅝	¾	Size of Angles. In.	Area, Four Angles. In.²	Distance Back to Back in Inches. 0	¼	5/16	⅜	½	⅝	¾
2½x2x3/16	3.24	2.0	2.5	2.6	2.7	3.0	3.3	3.7	3x2½x¼	5.24	5.2	6.2	6.5	6.7	7.3	7.9	8.6
" ¼	4.24	2.7	3.4	3.5	3.7	4.1	4.6	5.0	" 5/16	6.48	6.6	7.8	8.1	8.5	9.2	10.0	10.8
" 5/16	5.24	3.4	4.3	4.5	4.7	5.2	5.8	6.4	" ⅜	7.68	8.0	9.5	9.9	10.3	11.2	12.2	13.2
" ⅜	6.20	4.1	5.2	5.4	5.7	6.3	7.0	7.7	" 7/16	8.88	9.5	11.2	11.7	12.2	13.2	14.4	15.6
" 7/16	7.12	4.8	6.1	6.4	6.7	7.5	8.2	9.1	" ½	10.00	10.8	12.9	13.4	14.0	15.2	16.5	17.9
3½x2½x¼	5.76	5.2	6.2	6.5	6.8	7.4	8.0	8.7	3½x3x¼	6.24	9.0	10.4	10.7	11.1	11.9	12.7	13.6
" 5/16	7.12	6.6	7.9	8.3	8.6	9.4	10.2	11.0	" 5/16	7.72	11.4	13.1	13.5	14.0	15.0	16.0	17.2
" ⅜	8.44	8.0	9.6	10.0	10.4	11.3	12.3	13.4	" ⅜	9.20	13.8	15.8	16.3	16.9	18.1	19.4	20.8
" 7/16	9.72	9.4	11.2	11.7	12.2	13.3	14.5	15.7	" 7/16	10.60	16.0	18.4	19.1	19.8	21.2	22.7	24.3
" ½	11.00	10.8	12.9	13.5	14.1	15.4	16.7	18.2	" ½	12.00	18.6	21.4	22.2	23.0	24.6	26.4	28.2
4x3x¼	6.76	9.1	10.5	10.9	11.3	12.1	12.9	13.8	5x3x5/16	9.60	11.3	13.2	13.7	14.2	15.3	16.5	17.7
" 5/16	8.36	11.4	13.1	13.6	14.1	15.1	16.2	17.4	" ⅜	11.44	13.6	16.0	16.6	17.2	18.5	19.9	21.4
" ⅜	9.92	13.7	15.8	16.4	17.0	18.2	19.5	20.9	" 7/16	13.24	16.1	19.0	19.7	20.4	22.0	23.7	25.4
" 7/16	11.48	16.1	18.5	19.2	19.9	21.4	22.9	24.6	" ½	15.00	18.5	21.8	22.6	23.5	25.3	27.3	29.3
" ½	13.00	18.6	21.5	22.3	23.1	24.8	26.7	28.6	" 9/16	16.72	21.0	24.7	25.7	26.7	28.7	30.9	33.2
" 9/16	14.48	21.1	24.4	25.3	26.2	28.2	30.2	32.4	" ⅝	18.44	23.8	28.0	29.1	30.2	32.6	35.1	37.7
" ⅝	15.92	23.6	27.2	28.2	29.3	31.5	33.7	36.2	" 11/16	20.12	26.4	31.1	32.3	33.6	36.2	39.0	41.8

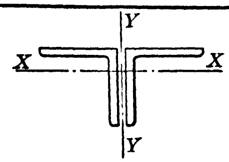
Size of Angles. In.	Area, Four Angles. In.²	Distance Back to Back of Angles in Inches. 0	¼	5/16	⅜	7/16	½	9/16	⅝	¾	⅞	1	1⅛	1¼
5x3½x5/16	10.24	18.1	20.4	21.0	21.7	22.4	23.0	23.7	24.5	26.0	....	....	....	....
" ⅜	12.20	21.7	24.6	25.3	26.1	26.9	27.8	28.6	29.5	31.3	....	....	....	....
" 7/16	14.12	25.5	28.8	29.7	30.6	31.6	32.5	33.6	34.6	36.8	....	....	....	....
" ½	16.00	29.4	33.3	34.4	35.5	36.6	37.7	38.9	40.1	42.6	....	....	....	....
" 9/16	17.88	33.3	37.7	38.9	40.1	41.4	42.7	44.0	45.4	48.3	....	....	....	....
" ⅝	19.68	37.1	42.1	43.4	44.8	46.2	47.7	49.2	50.7	53.9	....	....	....	....
" 11/16	21.48	41.0	46.6	48.0	49.6	51.2	52.8	54.4	56.1	59.7	....	....	....	....
" ¾	23.24	45.4	51.6	53.3	55.0	56.7	58.5	60.4	62.2	66.1	....	....	....	....
6x4x⅜	14.44	32.4	36.0	37.0	38.0	39.0	40.0	41.1	42.2	44.6	....	....	....	....
" 7/16	16.72	37.8	42.1	43.2	44.4	45.6	46.9	48.2	49.5	52.2	....	....	....	....
" ½	19.00	43.7	48.7	50.0	51.4	52.8	54.3	55.8	57.3	60.5	....	....	....	....
" 9/16	21.24	49.3	55.0	56.5	58.1	59.7	61.4	63.1	64.8	68.4	....	....	....	....
" ⅝	23.44	54.9	61.3	63.1	64.8	66.6	68.5	70.4	72.3	76.4	....	....	....	....
" 11/16	25.60	61.2	68.4	70.3	72.3	74.3	76.4	78.5	80.7	85.2	....	....	....	....
" ¾	27.76	67.1	75.0	77.1	79.3	81.5	83.8	86.2	88.5	93.5	....	....	....	....
" ⅞	31.92	78.9	88.5	91.0	93.6	96.2	98.9	101.7	104.5	110.3	....	....	....	....
" 1	36.00	92.1	103.4	106.3	109.3	112.4	115.6	118.8	122.1	128.9	....	....	....	....
8x6x7/16	23.72	126.9	....	....	140.6	143.0	145.5	148.1	150.7	156.0	161.5	167.2	173.0	179.1
" ½	27.00	145.1	....	....	160.9	163.7	166.6	169.5	172.5	178.6	184.9	191.5	198.3	205.2
" 9/16	30.24	164.2	....	....	182.3	185.5	188.8	192.1	195.5	202.5	209.7	217.2	224.8	232.7
" ⅝	33.44	182.6	....	....	202.8	206.4	210.1	213.8	217.6	225.4	233.5	241.8	250.4	259.2
" 11/16	36.60	201.0	....	....	223.5	227.4	231.5	235.6	239.8	248.5	257.4	266.5	276.0	285.8
" ¾	39.76	219.6	....	....	244.3	248.7	253.2	257.7	262.3	271.8	281.6	291.7	302.1	312.7
" ⅞	45.92	258.5	....	....	287.8	293.0	298.3	303.7	309.1	320.4	331.9	343.9	356.1	368.8
" 1	52.00	296.7	....	....	330.7	336.7	342.8	349.0	355.4	368.3	381.7	395.5	409.6	424.3

Radii of Gyration about Axis *Y–Y*, same as given in Table of Radii of Gyration of Two Angles.

TABLE 38.

Radii of Gyration of Two Angles with Equal Legs, Both Axes.

Radii of Gyration of Two Angles, Equal Legs.



For Distances Measured from Back to Back.

Size of Angles.	Area, Two Angles.	Axis X-X.	Axis Y-Y. Distance Back to Back in Inches.							Size of Angles.	Area, Two Angles.	Axis X-X.	Axis Y-Y. Distance Back to Back in Inches.						
In.	In.²		0	1/4	5/16	3/8	1/2	5/8	3/4	In.	In.²		0	1/4	5/16	3/8	1/2	5/8	3/4
2x2x3/16	1.42	.62	.84	.93	.95	.99	1.04	1.09	1.14	2½x2½x1/4	2.38	.77	1.05	1.14	1.17	1.19	1.24	1.29	1.34
" 1/4	1.88	.61	.85	.94	.96	.99	1.04	1.09	1.14	" 5/16	2.94	.76	1.06	1.15	1.17	1.20	1.25	1.30	1.35
" 5/16	2.30	.60	.86	.95	.98	1.00	1.05	1.10	1.15	" 3/8	3.46	.75	1.07	1.16	1.18	1.21	1.26	1.31	1.36
" 3/8	2.72	.59	.88	.97	.99	1.01	1.07	1.11	1.16	" 7/16	4.00	.75	1.08	1.17	1.20	1.22	1.27	1.32	1.37
3x3x1/4	2.88	.93	1.25	1.34	1.36	1.38	1.43	1.48	1.53	3½x3½x1/4	3.38	1.09	1.45	1.54	1.57	1.59	1.63	1.67	1.73
" 5/16	3.56	.92	1.26	1.36	1.38	1.40	1.45	1.50	1.55	" 5/16	4.18	1.08	1.47	1.56	1.58	1.60	1.65	1.69	1.74
" 3/8	4.22	.91	1.27	1.37	1.39	1.41	1.46	1.51	1.56	" 3/8	4.96	1.07	1.48	1.57	1.59	1.61	1.66	1.70	1.75
" 7/16	4.86	.91	1.28	1.38	1.40	1.42	1.47	1.52	1.57	" 7/16	5.74	1.07	1.49	1.58	1.60	1.62	1.67	1.72	1.77
" 1/2	5.50	.90	1.29	1.39	1.41	1.43	1.48	1.53	1.58	" 1/2	6.50	1.06	1.50	1.59	1.61	1.63	1.67	1.73	1.78
" 9/16	6.12	.89	1.30	1.40	1.42	1.45	1.50	1.54	1.60	" 9/16	7.24	1.05	1.51	1.60	1.62	1.64	1.69	1.75	1.80
" 5/8	6.72	.88	1.32	1.41	1.43	1.46	1.51	1.55	1.62	" 5/8	7.96	1.04	1.52	1.61	1.63	1.66	1.70	1.76	1.81

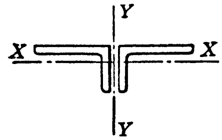
Size of Angles	Area, Two Angles.	Axis X-X.	Axis Y-Y. Distance Back to Back of Angles in Inches.												
In.	In.²		0	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	7/8	1	1 1/8	1 1/4
4x4x1/4	3.88	1.25	1.66	1.75	1.77	1.79	1.82	1.84	1.86	1.88	1.93	......	......	......	......
" 5/16	4.80	1.24	1.68	1.76	1.78	1.80	1.83	1.85	1.87	1.89	1.94	......	......	......	......
" 3/8	5.72	1.23	1.68	1.77	1.79	1.81	1.84	1.86	1.88	1.90	1.95	......	......	......	......
" 7/16	6.62	1.23	1.69	1.78	1.80	1.82	1.85	1.87	1.89	1.92	1.96	......	......	......	......
" 1/2	7.50	1.22	1.70	1.79	1.81	1.83	1.86	1.88	1.90	1.93	1.97	......	......	......	......
" 9/16	8.36	1.21	1.71	1.80	1.82	1.85	1.87	1.90	1.92	1.94	1.99	......	......	......	......
" 5/8	9.22	1.20	1.72	1.81	1.83	1.86	1.88	1.91	1.93	1.95	2.00	......	......	......	......
5x5x3/8	7.22	1.56	2.08	2.17	2.19	2.22	2.24	2.26	2.28	2.31	2.35	......	......	......	......
" 7/16	8.36	1.55	2.09	2.18	2.20	2.22	2.25	2.27	2.29	2.32	2.37	......	......	......	......
" 1/2	9.50	1.54	2.10	2.19	2.21	2.23	2.26	2.28	2.30	2.33	2.38	......	......	......	......
" 9/16	10.62	1.53	2.11	2.20	2.22	2.25	2.27	2.29	2.32	2.34	2.39	......	......	......	......
" 5/8	11.72	1.52	2.12	2.21	2.23	2.26	2.28	2.30	2.33	2.35	2.40	......	......	......	......
" 11/16	12.80	1.51	2.13	2.22	2.24	2.27	2.29	2.32	2.34	2.36	2.41	......	......	......	......
" 3/4	13.88	1.50	2.14	2.23	2.25	2.28	2.30	2.33	2.35	2.37	2.42	......	......	......	......
6x6x3/8	8.72	1.88	2.49	......	......	2.62	2.64	2.66	2.69	2.71	2.75	2.80	2.85	2.90	2.94
" 7/16	10.12	1.87	2.50	......	......	2.63	2.65	2.67	2.69	2.72	2.76	2.81	2.86	2.91	2.95
" 1/2	11.50	1.86	2.51	......	......	2.64	2.66	2.68	2.71	2.73	2.77	2.82	2.87	2.91	2.96
" 9/16	12.86	1.85	2.52	......	......	2.65	2.67	2.70	2.72	2.74	2.79	2.84	2.88	2.93	2.98
" 5/8	14.22	1.84	2.53	......	......	2.66	2.68	2.71	2.73	2.75	2.80	2.85	2.89	2.94	2.99
" 11/16	15.56	1.83	2.53	......	......	2.67	2.69	2.71	2.74	2.76	2.81	2.85	2.90	2.95	3.00
" 3/4	16.88	1.83	2.55	......	......	2.68	2.71	2.73	2.76	2.78	2.83	2.88	2.92	2.97	3.02
" 7/8	19.46	1.81	2.57	......	......	2.70	2.73	2.75	2.77	2.80	2.85	2.90	2.94	2.99	3.04
" 1	22.00	1.80	2.59	......	......	2.72	2.75	2.77	2.79	2.82	2.87	2.92	2.97	3.01	3.06
8x8x1/2	15.50	2.51	3.32	......	......	3.44	3.47	3.49	3.52	3.54	3.58	3.63	3.67	3.72	3.77
" 9/16	17.36	2.50	3.33	......	......	3.46	3.48	3.50	3.53	3.55	3.59	3.64	3.68	3.73	3.78
" 5/8	19.22	2.49	3.34	......	......	3.47	3.49	3.51	3.53	3.56	3.60	3.64	3.69	3.74	3.78
" 11/16	21.06	2.48	3.35	......	......	3.48	3.50	3.52	3.54	3.57	3.61	3.65	3.70	3.75	3.79
" 3/4	22.88	2.47	3.36	......	......	3.49	3.51	3.53	3.56	3.58	3.62	3.67	3.72	3.76	3.81
" 7/8	26.46	2.45	3.38	......	......	3.51	3.53	3.55	3.57	3.60	3.64	3.69	3.74	3.78	3.83
" 1	30.00	2.44	3.40	......	......	3.53	3.55	3.57	3.60	3.62	3.67	3.71	3.76	3.81	3.86
" 1 1/8	33.46	2.42	3.42	......	......	3.55	3.57	3.60	3.62	3.64	3.69	3.74	3.79	3.83	3.88

Moments of Inertia about Axis Y-Y equal one-half of values given in Table of Moments of Inertia of Four Angles, Table 35.

TABLE 39.

Radii of Gyration of Two Angles with Unequal Legs, Both Axes.

Long Legs Out.



Radii of Gyration of Two Angles, Long Legs Turned Out.

For Distances Measured from Back to Back.

Size of Angles.	Area, Two Angles.	Axis X-X.	Axis Y-Y. Distance Back to Back in Inches.							Size of Angles.	Area, Two Angles.	Axis X-X.	Axis Y-Y. Distance Back to Back in Inches.						
In.	In.²		0	¼	5/16	⅜	½	⅝	¾	In.	In.²		0	¼	5/16	⅜	½	⅝	¾
2½x2x3/16	1.62	.60	1.10	1.19	1.22	1.24	1.29	1.34	1.38	3x2½x¼	2.62	.75	1.31	1.40	1.42	1.45	1.50	1.55	1.59
" ¼	2.12	.59	1.11	1.20	1.23	1.25	1.30	1.36	1.40	" 5/16	3.24	.74	1.32	1.41	1.43	1.46	1.51	1.56	1.60
" 5/16	2.62	.58	1.12	1.21	1.24	1.26	1.31	1.37	1.42	" ⅜	3.84	.74	1.33	1.43	1.45	1.48	1.53	1.58	1.62
" ⅜	3.10	.58	1.13	1.22	1.25	1.28	1.32	1.38	1.44	" 7/16	4.44	.73	1.34	1.44	1.46	1.49	1.54	1.59	1.64
" 7/16	3.56	.57	1.14	1.24	1.26	1.29	1.33	1.39	1.46	" ½	5.00	.72	1.35	1.45	1.47	1.50	1.55	1.60	1.65
3½x2½x¼	2.88	.74	1.58	1.67	1.69	1.71	1.76	1.81	1.86	3½x3x¼	3.12	.91	1.52	1.61	1.63	1.65	1.70	1.75	1.79
" 5/16	3.56	.73	1.60	1.68	1.70	1.73	1.77	1.82	1.88	" 5/16	3.86	.90	1.52	1.61	1.64	1.66	1.71	1.76	1.81
" ⅜	4.22	.72	1.61	1.69	1.72	1.74	1.79	1.84	1.89	" ⅜	4.60	.90	1.53	1.62	1.65	1.67	1.72	1.77	1.82
" 7/16	4.86	.71	1.61	1.70	1.73	1.75	1.80	1.85	1.90	" 7/16	5.30	.89	1.54	1.63	1.66	1.68	1.73	1.78	1.83
" ½	5.50	.70	1.62	1.71	1.74	1.76	1.81	1.86	1.91	" ½	6.00	.88	1.55	1.65	1.68	1.70	1.75	1.80	1.85
4x3x¼	3.38	.89	1.77	1.87	1.89	1.92	1.96	2.01	2.06	5x3x5/16	4.80	.85	2.33	2.42	2.45	2.47	2.52	2.57	2.62
" 5/16	4.18	.89	1.79	1.88	1.90	1.93	1.97	2.02	2.07	" ⅜	5.72	.84	2.34	2.43	2.46	2.48	2.53	2.58	2.63
" ⅜	4.96	.88	1.80	1.89	1.91	1.94	1.98	2.03	2.08	" 7/16	6.62	.84	2.35	2.45	2.47	2.49	2.54	2.59	2.64
" 7/16	5.74	.87	1.81	1.90	1.92	1.95	1.99	2.04	2.09	" ½	7.50	.83	2.36	2.46	2.48	2.50	2.55	2.60	2.65
" ½	6.50	.86	1.82	1.92	1.94	1.96	2.01	2.06	2.11	" 9/16	8.36	.82	2.37	2.47	2.49	2.52	2.57	2.61	2.66
" 9/16	7.24	.86	1.83	1.93	1.95	1.97	2.02	2.07	2.12	" ⅝	9.22	.82	2.39	2.48	2.51	2.53	2.58	2.63	2.68
" ⅝	7.96	.85	1.84	1.94	1.96	1.98	2.03	2.08	2.14	" 11/16	10.06	.81	2.40	2.49	2.52	2.54	2.59	2.64	2.69

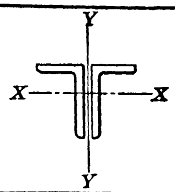
Size of Angles.	Area, Two Angles.	Axis X-X.	Axis Y-Y. Distance Back to Back of Angles in Inches.												
In.	In.²		0	¼	5/16	⅜	7/16	½	9/16	⅝	¾	⅞	1	1¼	1½
5x3½x5/16	5.12	1.03	2.26	2.35	2.37	2.39	2.42	2.44	2.47	2.49	2.54	......	......	......	......
" ⅜	6.10	1.02	2.27	2.36	2.38	2.40	2.43	2.45	2.48	2.50	2.55	......	......	......	......
" 7/16	7.06	1.01	2.28	2.37	2.39	2.41	2.44	2.46	2.49	2.52	2.56	......	......	......	......
" ½	8.00	1.01	2.29	2.38	2.41	2.43	2.45	2.48	2.50	2.53	2.58	......	......	......	......
" 9/16	8.94	1.00	2.30	2.39	2.42	2.44	2.46	2.49	2.51	2.54	2.59	......	......	......	......
" ⅝	9.84	.99	2.31	2.40	2.43	2.45	2.48	2.50	2.52	2.55	2.60	......	......	......	......
" 11/16	10.74	.98	2.32	2.41	2.44	2.46	2.49	2.51	2.53	2.56	2.61	......	......	......	......
" ¾	11.62	.98	2.33	2.43	2.46	2.48	2.51	2.53	2.55	2.58	2.63	......	......	......	......
6x4x⅜	7.22	1.17	2.74	2.83	2.85	2.87	2.90	2.92	2.94	2.97	3.01	......	......	......	......
" 7/16	8.36	1.16	2.75	2.84	2.86	2.88	2.91	2.93	2.95	2.98	3.02	......	......	......	......
" ½	9.50	1.15	2.76	2.85	2.88	2.90	2.92	2.95	2.97	2.99	3.04	......	......	......	......
" 9/16	10.62	1.14	2.77	2.86	2.88	2.91	2.93	2.96	2.98	3.00	3.05	......	......	......	......
" ⅝	11.72	1.13	2.78	2.87	2.89	2.92	2.94	2.97	2.99	3.01	3.06	......	......	......	......
" 11/16	12.80	1.13	2.79	2.89	2.91	2.94	2.96	2.98	3.01	3.03	3.08	......	......	......	......
" ¾	13.88	1.12	2.80	2.90	2.92	2.95	2.97	2.99	3.02	3.04	3.09	......	......	......	......
" ⅞	15.96	1.11	2.82	2.92	2.94	2.97	2.99	3.01	3.04	3.06	3.11	......	......	......	......
" 1	18.00	1.09	2.85	2.95	2.97	2.99	3.02	3.04	3.07	3.09	3.14	......	......	......	......
8x6x7/16	11.86	1.80	3.55	......	......	3.68	3.71	3.73	3.75	3.77	3.82	3.87	3.91	3.96	4.01
" ½	13.50	1.79	3.56	......	......	3.69	3.71	3.74	3.76	3.78	3.83	3.88	3.92	3.97	4.02
" 9/16	15.12	1.78	3.57	......	......	3.71	3.73	3.75	3.77	3.80	3.84	3.89	3.94	3.99	4.03
" ⅝	16.72	1.77	3.58	......	......	3.71	3.74	3.76	3.78	3.81	3.85	3.90	3.95	4.00	4.04
" 11/16	18.30	1.77	3.59	......	......	3.72	3.75	3.77	3.79	3.82	3.86	3.91	3.96	4.01	4.05
" ¾	19.88	1.76	3.60	......	......	3.73	3.76	3.78	3.80	3.82	3.87	3.92	3.97	4.02	4.06
" ⅞	22.96	1.74	3.62	......	......	3.76	3.78	3.81	3.83	3.85	3.90	3.95	3.99	4.04	4.09
" 1	26.00	1.73	3.64	......	......	3.78	3.80	3.82	3.85	3.87	3.92	3.97	4.02	4.07	4.12

Moments of Inertia about Axis Y-Y equal one-half of values given in Table of Moments of Inertia of Four Angles, Table 36.

TABLE 40.

Radii of Gyration of Two Angles with Unequal Legs, Both Axes. Short Legs Out.

Radii of Gyration of Two Angles, Short Legs Turned Out.



For Distances Measured from Back to Back.

Size of Angles.	Area Two Angles.	Axis X-X.	Axis Y-Y. Distance Back to Back in Inches.							Size of Angles.	Area Two Angles.	Axis X-X.	Axis Y-Y. Distance Back to Back in Inches.						
In.	In.²		0	¼	5/16	⅜	½	⅝	¾	In.	In.²		0	¼	5/16	⅜	½	⅝	¾
2½x2x3/16	1.62	.79	.79	.88	.90	.92	.96	1.02	1.07	3x2½x¼	2.62	.95	1.00	1.09	1.11	1.13	1.18	1.23	1.28
" ¼	2.12	.78	.80	.89	.91	.93	.98	1.04	1.09	" 5/16	3.24	.94	1.01	1.10	1.12	1.14	1.19	1.24	1.29
" 5/16	2.62	.78	.81	.91	.93	.95	1.00	1.05	1.10	" ⅜	3.84	.93	1.02	1.11	1.14	1.16	1.21	1.26	1.31
" ⅜	3.10	.77	.81	.92	.94	.96	1.01	1.06	1.11	" 7/16	4.44	.92	1.03	1.12	1.15	1.17	1.22	1.27	1.33
" 7/16	3.56	.76	.82	.93	.95	.97	1.02	1.07	1.13	" ½	5.00	.91	1.04	1.14	1.16	1.18	1.23	1.28	1.34
3½x2½x¼	2.88	1.12	.95	1.04	1.06	1.09	1.13	1.18	1.23	3½x3x¼	3.12	1.11	1.20	1.29	1.31	1.33	1.38	1.43	1.48
" 5/16	3.56	1.11	.96	1.05	1.08	1.10	1.15	1.20	1.24	" 5/16	3.86	1.10	1.22	1.30	1.32	1.35	1.39	1.44	1.49
" ⅜	4.22	1.10	.97	1.07	1.09	1.11	1.16	1.21	1.26	" ⅜	4.60	1.09	1.23	1.31	1.33	1.36	1.40	1.45	1.50
" 7/16	4.86	1.09	.98	1.07	1.10	1.12	1.17	1.22	1.27	" 7/16	5.30	1.08	1.23	1.32	1.34	1.37	1.41	1.46	1.51
" ½	5.50	1.09	.99	1.08	1.11	1.13	1.18	1.23	1.29	" ½	6.00	1.07	1.24	1.33	1.36	1.39	1.43	1.48	1.53
4x3x¼	3.38	1.28	1.16	1.24	1.27	1.29	1.34	1.38	1.43	5x3x5/16	4.80	1.61	1.09	1.17	1.20	1.22	1.26	1.31	1.36
" 5/16	4.18	1.27	1.17	1.25	1.28	1.30	1.35	1.39	1.44	" ⅜	5.72	1.61	1.09	1.18	1.21	1.23	1.27	1.32	1.37
" ⅜	4.96	1.26	1.17	1.26	1.28	1.31	1.36	1.40	1.45	" 7/16	6.62	1.60	1.10	1.20	1.22	1.24	1.29	1.34	1.39
" 7/16	5.74	1.25	1.18	1.27	1.29	1.32	1.36	1.41	1.46	" ½	7.50	1.59	1.11	1.21	1.23	1.25	1.30	1.35	1.40
" ½	6.50	1.25	1.20	1.28	1.31	1.33	1.38	1.43	1.48	" 9/16	8.36	1.58	1.12	1.22	1.24	1.26	1.31	1.36	1.41
" 9/16	7.24	1.24	1.21	1.30	1.32	1.35	1.40	1.45	1.50	" ⅝	9.22	1.57	1.14	1.23	1.26	1.28	1.33	1.38	1.43
" ⅝	7.96	1.23	1.22	1.31	1.33	1.36	1.41	1.46	1.51	" 11/16	10.06	1.56	1.15	1.24	1.27	1.29	1.34	1.39	1.44

Size of Angles.	Area Two Angles.	Axis X-X.	Axis Y-Y. Distance Back to Back of Angles in Inches.												
In.	In.²		0	¼	5/16	⅜	7/16	½	9/16	⅝	¾	⅞	1	1⅛	1¼
5x3½x5/16	5.12	1.61	1.33	1.41	1.43	1.46	1.48	1.50	1.52	1.55	1.59	......	......	......	......
" ⅜	6.10	1.60	1.34	1.42	1.44	1.46	1.49	1.51	1.53	1.56	1.60	......	......	......	......
" 7/16	7.06	1.59	1.35	1.43	1.45	1.47	1.50	1.52	1.54	1.57	1.62	......	......	......	......
" ½	8.00	1.58	1.36	1.44	1.47	1.49	1.51	1.54	1.56	1.58	1.63	......	......	......	......
" 9/16	8.94	1.57	1.37	1.45	1.48	1.50	1.52	1.55	1.57	1.59	1.64	......	......	......	......
" ⅝	9.84	1.56	1.38	1.46	1.49	1.51	1.53	1.56	1.58	1.60	1.66	......	......	......	......
" 11/16	10.74	1.56	1.38	1.47	1.50	1.52	1.54	1.57	1.59	1.62	1.67	......	......	......	......
" ¾	11.62	1.55	1.40	1.49	1.51	1.54	1.56	1.59	1.61	1.63	1.69	......	......	......	......
6x4x⅜	7.22	1.93	1.50	1.58	1.60	1.62	1.64	1.66	1.69	1.71	1.76	......	......	......	......
" 7/16	8.36	1.92	1.50	1.59	1.61	1.63	1.66	1.68	1.70	1.72	1.77	......	......	......	......
" ½	9.50	1.91	1.51	1.60	1.62	1.65	1.67	1.69	1.71	1.74	1.78	......	......	......	......
" 9/16	10.62	1.90	1.52	1.61	1.63	1.66	1.68	1.70	1.72	1.75	1.79	......	......	......	......
" ⅝	11.72	1.90	1.53	1.62	1.64	1.67	1.69	1.71	1.73	1.76	1.81	......	......	......	......
" 11/16	12.80	1.89	1.55	1.63	1.66	1.68	1.71	1.73	1.75	1.77	1.82	......	......	......	......
" ¾	13.88	1.88	1.56	1.64	1.67	1.69	1.72	1.74	1.76	1.79	1.84	......	......	......	......
" ⅞	15.96	1.86	1.58	1.66	1.69	1.71	1.74	1.76	1.79	1.81	1.86	......	......	......	......
" 1	18.00	1.85	1.60	1.69	1.72	1.74	1.77	1.79	1.82	1.84	1.89	......	......	......	......
8x6x7/16	11.86	2.57	2.31	......	......	2.43	2.45	2.47	2.49	2.52	2.56	2.61	2.66	2.70	2.75
" ½	13.50	2.56	2.32	......	......	2.44	2.46	2.48	2.51	2.53	2.57	2.62	2.66	2.71	2.76
" 9/16	15.12	2.55	2.33	......	......	2.46	2.48	2.50	2.52	2.54	2.59	2.63	2.68	2.73	2.77
" ⅝	16.72	2.54	2.34	......	......	2.46	2.49	2.51	2.53	2.55	2.60	2.64	2.69	2.74	2.79
" 11/16	18.30	2.54	2.34	......	......	2.47	2.49	2.52	2.54	2.56	2.61	2.65	2.70	2.75	2.80
" ¾	19.88	2.53	2.35	......	......	2.48	2.50	2.52	2.55	2.57	2.62	2.66	2.71	2.77	2.81
" ⅞	22.96	2.51	2.37	......	......	2.51	2.53	2.55	2.57	2.59	2.64	2.69	2.74	2.79	2.83
" 1	26.00	2.49	2.39	......	......	2.52	2.54	2.57	2.59	2.62	2.66	2.71	2.76	2.81	2.86

Moments of Inertia about Axis *Y–Y* equal one-half of values given in Table of Moments of Inertia of Four Angles, Table 37.

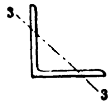
TABLE 41

Safe Loads of Single Angle Struts

Equal Leg Angles

American Bridge Company Standards

Safe loads in thousands of pounds for least radius of gyration
$p = 16{,}000 - 70\ l/r$



To left of heavy line values of l/r do not exceed 125
To right of heavy line values of l/r do not exceed 150

Size	Thickness	Length in Feet												
Inches	Inches	3	4	5	6	7	8	9	10	11	12	13	14	15
$1\frac{1}{2} \times 1\frac{1}{2}$	$\frac{3}{16}$	4	....	....	....	....	....	....	....	....	....	....	....	....
$1\frac{3}{4} \times 1\frac{3}{4}$	$\frac{3}{16}$	5	4	....	....	....	....	....	....	....	....	....	....	....
	$\frac{1}{4}$	7	5	....	....	....	....	....	....	....	....	....	....	....
2×2	$\frac{3}{16}$	7	5	4	....	....	....	....	....	....	....	....	....	....
	$\frac{1}{4}$	9	7	....	....	....	....	....	....	....	....	....	....	....
	$\frac{5}{16}$	11	8	....	....	....	....	....	....	....	....	....	....	....
$2\frac{1}{2} \times 2\frac{1}{2}$	$\frac{3}{16}$	10	8	7	5	....	....	....	....	....	....	....	....	....
	$\frac{1}{4}$	13	11	9	7	....	....	....	....	....	....	....	....	....
	$\frac{5}{16}$	16	13	11	8	....	....	....	....	....	....	....	....	....
3×3	$\frac{1}{4}$	17	15	13	11	9	....	....	....	....	....	....	....	....
	$\frac{5}{16}$	21	18	16	13	11	....	....	....	....	....	....	....	....
	$\frac{3}{8}$	25	22	18	15	12	....	....	....	....	....	....	....	....
	$\frac{7}{16}$	28	25	21	18	14	....	....	....	....	....	....	....	....
$3\frac{1}{2} \times 3\frac{1}{2}$	$\frac{5}{16}$	26	23	21	18	16	13	....	....	....	....	....	....	....
	$\frac{3}{8}$	31	28	25	22	19	16	....	....	....	....	....	....	....
	$\frac{7}{16}$	35	32	28	25	21	18	....	....	....	....	....	....	....
4×4	$\frac{5}{16}$	31	28	26	23	21	18	15	....	....	....	....	....	....
	$\frac{3}{8}$	37	34	31	27	24	21	18	....	....	....	....	....	....
	$\frac{7}{16}$	42	39	35	32	28	24	21	....	....	....	....	....	....
	$\frac{1}{2}$	48	44	40	36	32	28	24	....	....	....	....	....	....
5×5	$\frac{3}{8}$	49	46	42	39	36	33	30	27	24	21	....	....	....
	$\frac{7}{16}$	56	53	49	45	42	38	35	31	27	24	....	....	....
	$\frac{1}{2}$	64	60	56	52	47	43	39	35	31	27	....	....	....
	$\frac{9}{16}$	71	67	62	58	53	48	44	39	35	30	....	....	....
6×6	$\frac{3}{8}$	60	57	54	51	48	45	42	39	36	33	30	27	....
	$\frac{7}{16}$	70	67	63	59	56	52	49	45	42	38	34	31	....
	$\frac{1}{2}$	80	76	72	67	63	59	55	51	47	43	39	35	....
	$\frac{9}{16}$	89	85	80	75	71	66	62	57	53	48	43	39	....
	$\frac{5}{8}$	98	93	89	83	78	73	68	63	58	53	48	43	....

Note: The values in this table have been calculated on the assumption that the angle is fastened by both legs.—M. S. K.

TABLE 42

SAFE LOADS OF SINGLE ANGLE STRUTS
UNEQUAL LEG ANGLES
AMERICAN BRIDGE COMPANY STANDARDS

Safe loads in thousands of pounds for least radius of gyration
$p = 16{,}000 - 70\ l/r$

To left of heavy line values of l/r do not exceed 125
To right of heavy line values of l/r do not exceed 150

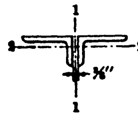
Size	Thickness	Length in Feet										
Inches	Inches	3	4	5	6	7	8	9	10	11	12	13
$2 \times 1\frac{3}{8}$	$\frac{3}{16}$	5	...	...	...	...	...	...	...	...	...	...
$2\frac{1}{2} \times 2$	$\frac{3}{16}$	8	7	5	...	...	...	...	...	...	...	...
	$\frac{1}{4}$	11	8	6	...	...	...	...	...	...	...	...
	$\frac{5}{16}$	13	10	8	...	...	...	...	...	...	...	...
3×2	$\frac{1}{4}$	12	10	7	...	...	...	...	...	...	...	...
	$\frac{5}{16}$	15	12	9	...	...	...	...	...	...	...	...
$3 \times 2\frac{1}{2}$	$\frac{1}{4}$	15	13	11	8	...	...	...	...	...	...	...
	$\frac{5}{16}$	18	16	13	11	...	...	...	...	...	...	...
	$\frac{3}{8}$	21	18	15	12	...	...	...	...	...	...	...
$3\frac{1}{2} \times 2\frac{1}{2}$	$\frac{1}{4}$	16	14	12	10	...	...	...	...	...	...	...
	$\frac{5}{16}$	20	17	15	12	...	...	...	...	...	...	...
	$\frac{3}{8}$	24	21	17	14	...	...	...	...	...	...	...
$3\frac{1}{2} \times 3$	$\frac{5}{16}$	23	21	18	15	13	...	...	...	...	...	...
	$\frac{3}{8}$	27	24	21	18	15	...	...	...	...	...	...
	$\frac{7}{16}$	32	28	24	21	17	...	...	...	...	...	...
	$\frac{1}{2}$	36	32	28	24	20	...	...	...	...	...	...
4×3	$\frac{5}{16}$	25	23	20	17	15	12	...	...	...	...	...
	$\frac{3}{8}$	30	27	23	20	17	14	...	...	...	...	...
	$\frac{7}{16}$	35	31	27	23	20	16	...	...	...	...	...
	$\frac{1}{2}$	39	35	31	26	22	18	...	...	...	...	...
$5 \times 3\frac{1}{2}$	$\frac{5}{16}$	32	30	27	24	21	18	15	...	...	...	...
	$\frac{3}{8}$	39	35	32	29	25	22	18	...	...	...	...
	$\frac{7}{16}$	45	41	37	33	29	25	21	...	...	...	...
	$\frac{1}{2}$	50	46	42	37	33	28	24	...	...	...	...
6×4	$\frac{3}{8}$	47	44	41	37	34	30	27	23	20	...	...
	$\frac{7}{16}$	55	51	47	43	39	35	31	26	...	...	...
	$\frac{1}{2}$	62	58	53	49	44	39	35	30	...	...	...
	$\frac{9}{16}$	70	65	59	54	49	44	39	34	...	...	...
	$\frac{5}{8}$	77	71	65	59	54	48	42	36	...	...	...

Note: The values in this table have been calculated on the assumption that the angle is fastened by both legs.—M. S. K.

TABLE 43

Safe Loads of Two Angle Struts, Axis 1–1
Equal Leg, and Unequal Leg with Long Leg Turned Out
American Bridge Company Standards

Safe loads in thousands of pounds with respect to axis 1–1
$p = 16{,}000 - 70\ l/r$



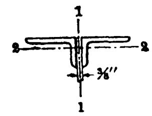
To left of heavy line values of l/r do not exceed 125
To right of heavy line values of l/r do not exceed 150

Size of Angles	Thickness	Radius of Gyration	Weight of Two Angles per Foot	Area of Two Angles	Length in Feet																		
In.	In.	In.	Lb.	In.²	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
2 ×2	$\frac{3}{16}$	.98	5.0	1.44	16	14	13	12	11	9	8	...	...	...	...	...	...	...	...	...	...	...	...
	$\frac{1}{4}$	.99	6.4	1.88	21	19	17	16	14	13	11	...	...	...	...	...	...	...	...	...	...	...	...
2½×2	$\frac{3}{16}$	1.24	5.6	1.62	19	18	17	16	15	14	13	12	11	9	...	...	...	...	...	...	...	...	...
	$\frac{1}{4}$	1.25	7.4	2.12	25	24	23	21	20	18	17	15	14	13	...	...	...	...	...	...	...	...	...
	$\frac{5}{16}$	1.26	9.0	2.62	32	30	28	26	25	23	21	19	18	16	...	...	...	...	...	...	...	...	...
2½×2½	$\frac{1}{4}$	1.19	8.2	2.38	28	26	25	23	21	20	18	16	15	...	...	...	...	...	...	...	...	...	...
	$\frac{5}{16}$	1.20	10.0	2.94	35	33	31	29	26	24	22	20	18	16	...	...	...	...	...	...	...	...	...
3 ×2	$\frac{1}{4}$	1.52	8.2	2.38	30	29	27	26	25	24	22	21	20	18	17	16	14	...	...	...	...	...	...
	$\frac{5}{16}$	1.53	10.0	2.94	37	36	34	33	31	29	28	26	25	23	21	20	18	17	...	...	...	...	...
	$\frac{3}{8}$	1.55	11.8	3.46	44	42	40	38	37	35	33	31	29	27	25	23	22	20	...	...	...	...	...
3 ×2½	$\frac{1}{4}$	1.45	9.0	2.62	33	31	30	28	27	25	24	22	21	19	18	16	15	...	...	...	...	...	...
	$\frac{5}{16}$	1.46	11.2	3.24	41	39	37	35	33	31	29	28	26	24	22	20	18	...	...	...	...	...	...
	$\frac{3}{8}$	1.48	13.2	3.84	48	46	44	42	40	37	35	33	31	29	27	24	22	...	...	...	...	...	...
3 ×3	$\frac{1}{4}$	1.39	9.8	2.88	36	34	32	30	29	27	25	23	22	20	18	17	...	...	...	...	...	...	...
	$\frac{5}{16}$	1.40	12.2	3.56	44	42	40	38	36	33	31	29	27	25	23	21	...	...	...	...	...	...	...
	$\frac{3}{8}$	1.41	14.4	4.22	52	50	47	45	42	40	37	35	32	30	27	25	...	...	...	...	...	...	...
	$\frac{7}{16}$	1.42	16.6	4.86	61	58	55	52	49	46	43	40	38	35	32	29	...	...	...	...	...	...	...
	$\frac{1}{2}$	1.44	18.8	5.50	69	66	62	59	56	53	49	46	43	40	37	33	30	...	...	...	...	...	...
3½×2½	$\frac{1}{4}$	1.71	9.8	2.88	38	36	35	33	32	31	29	28	26	25	23	22	21	19	18	16	...	...	...
	$\frac{5}{16}$	1.73	12.2	3.56	47	45	43	41	40	38	36	34	33	31	29	28	26	24	22	21	...	...	...
	$\frac{3}{8}$	1.74	14.4	4.22	55	53	51	49	47	45	43	41	39	37	35	33	31	29	27	25	...	...	...
	$\frac{7}{16}$	1.76	16.6	4.86	64	62	59	57	55	52	50	48	45	43	41	38	36	34	31	29	27	...	...
	$\frac{1}{2}$	1.77	18.8	5.50	72	70	67	65	62	59	57	54	51	49	46	44	41	38	36	33	31	...	...
3½×3	$\frac{5}{16}$	1.66	13.2	3.86	50	48	46	44	42	40	38	36	34	32	30	28	27	25	23	...	...	...	...
	$\frac{3}{8}$	1.67	15.8	4.60	60	57	55	53	50	48	46	44	41	39	37	34	32	30	27	...	...	...	...
	$\frac{7}{16}$	1.69	18.2	5.30	69	66	64	61	58	56	53	50	48	45	42	40	37	34	32	29	...	...	...
	$\frac{1}{2}$	1.70	20.4	6.00	78	75	72	69	66	63	60	58	54	52	49	46	43	40	37	34	...	...	...
3½×3½	$\frac{5}{16}$	1.60	14.4	4.18	54	52	49	47	45	43	41	38	36	34	32	30	27	25	23	...	...	...	...
	$\frac{3}{8}$	1.61	17.0	4.96	64	61	59	56	53	51	48	46	43	41	38	35	33	30	28	...	...	...	...
	$\frac{7}{16}$	1.63	19.6	5.74	74	71	68	65	62	59	56	53	50	47	45	42	39	36	33	...	...	...	...
	$\frac{1}{2}$	1.64	22.2	6.50	84	81	77	74	71	67	64	61	57	54	51	47	44	40	37	...	...	...	...
4 ×3	$\frac{5}{16}$	1.93	14.4	4.18	56	54	52	51	49	47	45	43	41	40	38	36	34	32	30	29	27	25	23
	$\frac{3}{8}$	1.94	17.0	4.96	66	64	62	60	58	56	54	51	49	47	45	43	41	39	36	34	32	30	28
	$\frac{7}{16}$	1.95	19.6	5.74	77	75	72	70	67	65	62	60	57	55	52	50	47	45	42	40	37	35	32
	$\frac{1}{2}$	1.96	22.2	6.50	87	85	82	79	76	73	71	68	65	62	59	56	54	51	48	45	43	40	37
	$\frac{9}{16}$	1.97	24.8	7.24	97	94	91	88	85	82	79	76	73	70	67	64	61	57	54	51	48	45	42
	$\frac{5}{8}$	1.99	27.2	7.96	107	104	100	97	94	90	87	84	80	77	74	70	67	64	60	57	53	50	47

TABLE 43.—*Continued*

SAFE LOADS OF TWO ANGLE STRUTS, AXIS 1–1
EQUAL LEG, AND UNEQUAL LEG WITH LONG LEG TURNED OUT
AMERICAN BRIDGE COMPANY STANDARDS

Safe loads in thousands of pounds with respect to axis 1–1
$p = 16{,}000 - 70\ l/r$



To left of heavy line values of l/r do not exceed 125
To right of heavy line values of l/r do not exceed 150

Size of Angles	Thickness	Radius of Gyration	Weight of Two Angles per Foot	Area of Two Angles	Length in Feet																			
In.	In.	In.	Lb.	In.²	6	7	8	9	10	11	12	13	14	16	18	20	22	24	26	28	30	32	34	36
4×4	5/16	1.80	16.4	4.80	63	61	59	57	54	52	50	48	45	41	36	32	28	...	...	...	...	...	...	...
	3/8	1.81	19.6	5.72	76	73	70	68	65	62	60	57	54	49	44	38	33	...	...	...	...	...	...	...
	7/16	1.83	22.6	6.62	88	85	81	78	75	72	69	66	63	57	51	45	39	...	...	...	...	...	...	...
	1/2	1.84	25.6	7.50	99	96	93	89	86	82	79	75	72	65	58	52	45	...	...	...	...	...	...	...
5×3	5/16	2.47	16.4	4.80	67	65	64	62	61	59	57	56	54	51	47	44	41	38	34	31	28	...	...	...
	3/8	2.48	19.6	5.72	80	78	76	74	72	70	68	66	64	61	57	53	49	45	41	37	33	...	...	...
	7/16	2.49	22.6	6.62	93	90	88	86	84	81	79	77	75	71	66	61	57	52	48	44	39	...	...	...
	1/2	2.50	25.6	7.50	104	102	100	97	95	92	90	87	85	80	75	70	65	60	55	50	45	...	...	...
5×3½	5/16	2.40	17.4	5.12	71	69	68	66	64	62	60	59	57	53	50	46	42	39	35	32	28	...	...	...
	3/8	2.40	20.8	6.10	85	83	81	79	76	74	72	70	68	64	59	55	51	46	42	38	34	...	...	...
	7/16	2.41	24.0	7.06	98	96	93	91	89	86	84	81	79	74	69	64	59	54	49	44	40	...	...	...
	1/2	2.43	27.2	8.00	112	109	106	103	100	98	95	92	89	84	78	73	67	62	56	51	45	...	...	...
	9/16	2.44	30.4	8.94	125	122	119	115	112	109	105	103	100	94	88	81	75	69	63	57	51	...	...	...
	5/8	2.45	33.6	9.84	137	134	131	127	124	120	117	114	110	104	97	90	83	76	70	63	56	...	...	...
	11/16	2.46	36.6	10.74	150	146	143	139	135	132	128	124	121	114	106	99	91	84	77	70	62	...	...	...
	3/4	2.48	39.6	11.62	162	158	155	151	147	143	139	135	131	123	115	107	99	92	84	76	68	...	...	...
5×5	3/8	2.22	24.6	7.22	99	97	94	91	88	86	83	80	77	72	66	61	55	50	44	...	...	...	...	...
	7/16	2.23	28.6	8.36	115	112	109	105	102	99	96	93	90	83	77	71	64	58	52	...	...	...	...	...
	1/2	2.24	32.4	9.50	131	127	124	120	116	112	109	106	102	95	88	81	74	66	59	52	...	...	...	...
6×3½	3/8	2.95	23.4	6.84	98	96	94	92	90	88	86	84	82	78	74	71	67	63	59	55	51	47	43	39
	7/16	2.96	27.0	7.94	114	111	109	107	105	102	100	98	96	91	87	82	78	73	68	64	60	55	50	46
	1/2	2.98	30.6	9.00	129	126	124	121	119	116	114	111	108	103	98	93	88	83	78	73	68	63	57	52
	5/8	3.00	37.8	11.10	159	156	153	150	146	143	140	137	134	128	122	115	109	103	97	91	84	78	72	66
6×4	3/8	2.87	24.6	7.22	103	101	99	97	94	92	90	88	86	82	78	73	69	65	61	56	52	48	44	...
	7/16	2.88	28.6	8.36	119	117	114	112	109	107	105	102	100	95	90	85	80	75	70	65	61	56	51	46
	1/2	2.90	32.4	9.50	136	133	130	127	124	122	119	116	113	108	102	97	91	86	80	75	69	64	58	53
	9/16	2.91	36.2	10.62	152	149	145	142	139	136	133	130	127	121	115	109	102	96	90	84	78	72	65	59
	5/8	2.92	40.0	11.72	167	164	161	157	154	151	147	144	140	134	127	120	113	107	100	93	86	80	73	66
	11/16	2.93	43.6	12.82	183	179	176	172	169	165	161	158	154	147	139	131	124	116	109	102	95	87	80	72
	3/4	2.94	47.2	13.88	198	195	191	187	183	179	175	171	167	159	151	143	135	127	119	111	103	95	87	79
6×6	3/8	2.62	29.8	8.72	123	120	117	114	112	109	106	103	100	95	89	84	78	72	67	61	56	50	...	...
	7/16	2.63	34.4	10.12	143	139	136	133	130	126	123	120	117	110	104	97	91	84	78	72	65	59	...	...
	1/2	2.64	39.2	11.50	162	159	155	151	148	144	140	136	133	126	118	111	103	96	80	82	74	67	...	...
	9/16	2.65	43.8	12.86	181	177	173	169	165	161	157	153	149	141	132	124	116	108	100	92	84	75	...	...
	5/8	2.66	48.4	14.22	201	196	192	187	183	178	174	169	165	156	147	138	129	120	111	102	93	84	...	...
	11/16	2.67	53.0	15.56	220	215	210	205	200	195	190	186	181	171	161	151	141	132	122	112	102	92	...	...
	3/4	2.68	57.4	16.88	239	233	228	223	218	212	207	202	196	186	175	164	154	143	133	122	111	101	...	...

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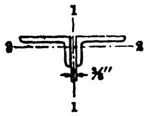
TABLE 44

Safe Loads of Two Angle Struts, Axis 2–2

Equal Leg, and Unequal Leg with Long Leg Turned Out

American Bridge Company Standards

Safe loads in thousands of pounds with respect to axis 2–2
$p = 16{,}000 - 70\ l/r$



To left of heavy line values of l/r do not exceed 125
To right of heavy line values of l/r do not exceed 150

Section Modulus S_2	Radius of Gyration r_2	Radius of Gyration r_1	Weight of Two Angles per Foot	Area of Two Angles	Thickness	Length in Feet											
In.³	In.	In.	Lb.	In.²	In.	3	4	5	6	7	8	9	10	11	12	13	14
						2″×2″ Angles											
.38	.62	.98	5.0	1.44	3/16	17	15	13	11	9	...	...	...	...	...	...	...
.50	.61	.99	6.4	1.88	1/4	22	20	17	15	12	...	...	...	...	...	...	...
						2½″×2″ Angles											
.40	.60	1.24	5.6	1.62	3/16	19	17	15	12	10	...	...	...	...	...	...	...
.50	.59	1.25	7.4	2.12	1/4	25	22	19	16	13	...	...	...	...	...	...	...
.62	.58	1.26	9.0	2.62	5/16	31	27	23	19	15	...	...	...	...	...	...	...
						2½″×2½″ Angles											
.80	.77	1.19	8.2	2.38	1/4	30	28	25	22	20	17	15	...	...	...	...	...
.96	.76	1.20	10.0	2.94	5/16	37	34	31	28	24	21	18	...	...	...	...	...
						3″×2″ Angles											
.50	.57	1.52	8.2	2.38	1/4	28	24	21	17	14	...	...	...	...	...	...	...
.64	.57	1.53	10.0	2.94	5/16	34	30	25	21	17	...	...	...	...	...	...	...
.74	.56	1.55	11.8	3.46	3/8	40	35	29	24	19	...	...	...	...	...	...	...
						3″×2½″ Angles											
.80	.75	1.45	9.0	2.62	1/4	33	30	27	24	21	18	16	...	...	...	...	...
.98	.74	1.46	11.2	3.24	5/16	41	37	33	30	26	22	19	...	...	...	...	...
1.16	.74	1.48	13.2	3.84	3/8	48	44	40	35	31	27	22	...	...	...	...	...
						3″×3″ Angles											
1.16	.93	1.39	9.8	2.88	1/4	38	36	33	30	28	25	22	20	17	...	...	...
1.42	.92	1.40	12.2	3.56	5/16	47	44	41	37	34	31	28	24	21	...	...	...
1.66	.91	1.41	14.4	4.22	3/8	56	52	48	44	40	36	32	29	25	...	...	...
1.90	.91	1.42	16.6	4.86	7/16	64	60	55	50	46	42	37	33	28	...	...	...
2.14	.90	1.44	18.8	5.50	1/2	73	67	62	57	52	47	42	37	32	...	...	...
						3½″×2½″ Angles											
.82	.74	1.71	9.8	2.88	1/4	36	33	30	26	23	20	17	...	...	...	...	...
1.00	.73	1.73	12.2	3.56	5/16	45	41	36	32	28	24	20	...	...	...	...	...
1.18	.72	1.74	14.4	4.22	3/8	53	48	43	38	33	28	23	...	...	...	...	...
1.36	.71	1.76	16.6	4.86	7/16	61	55	49	43	38	32	...	...	...	...	...	...
1.52	.70	1.77	18.8	5.50	1/2	68	62	55	48	42	35	...	...	...	...	...	...
						3½″×3″ Angles											
1.44	.90	1.66	13.2	3.86	5/16	51	47	44	40	37	33	29	26	22	...	...	...
1.70	.90	1.67	15.8	4.60	3/8	61	56	52	48	44	39	35	31	26	...	...	...
1.96	.89	1.69	18.2	5.30	7/16	70	65	60	55	50	45	40	35	30	...	...	...
2.20	.88	1.70	20.4	6.00	1/2	79	73	67	62	56	50	44	39	33	...	...	...

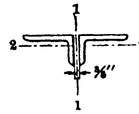
TABLE 44.—*Continued*

Safe Loads of Two Angle Struts, Axis 2–2

Equal Leg, and Unequal Leg with Long Leg Turned Out

American Bridge Company Standards

Safe loads in thousands of pounds with respect to axis 2–2
$p = 16{,}000 - 70\ l/r$



To left of heavy line values of l/r do not exceed 125
To right of heavy line values of l/r do not exceed 150

Section Modulus S_2	Radius of Gyration r_2	Radius of Gyration r_1	Weight of Two Angles per Ft.	Area of Two Angles	Thickness	Length in Feet 3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
In.^{3}	In.	In.	Lb.	In.^{2}	In.																	
						3½″×3½″ Angles																
1.96	1.08	1.60	14.4	4.18	5/16	57	54	51	47	44	41	38	35	31	28	25	..	..	..	..	..	..
2.30	1.07	1.61	17.0	4.96	3/8	68	64	60	56	52	48	44	40	37	33	29	..	..	..	..	..	..
2.64	1.07	1.63	19.6	5.74	7/16	78	74	69	65	60	56	51	47	42	38	33	..	..	..	..	..	..
2.98	1.06	1.64	22.2	6.50	1/2	89	83	78	73	68	63	58	53	47	42	37	..	..	..	..	..	..
						4″×3″ Angles																
1.48	.89	1.93	14.4	4.18	5/16	55	51	47	43	39	35	31	27	23	..	..	..	..	..	..	..	..
1.74	.88	1.94	17.0	4.96	3/8	65	60	56	51	46	41	37	32	27	..	..	..	..	..	..	..	..
1.98	.87	1.95	19.6	5.74	7/16	75	70	64	59	53	48	42	36	..	..	..	..	..	..	..	..	..
2.24	.86	1.96	22.2	6.50	1/2	85	79	72	66	60	53	47	40	..	..	..	..	..	..	..	..	..
2.46	.86	1.97	24.8	7.24	9/16	95	88	81	73	66	59	52	45	..	..	..	..	..	..	..	..	..
2.70	.85	1.99	27.2	7.96	5/8	104	96	88	80	72	64	57	49	..	..	..	..	..	..	..	..	..
						4″×4″ Angles																
2.58	1.24	1.80	16.4	4.80	5/16	67	64	61	57	54	51	48	44	41	38	35	31	28	..	..	..	..
3.04	1.23	1.81	19.6	5.72	3/8	80	76	72	68	64	60	56	53	49	45	41	37	33	..	..	..	..
3.50	1.23	1.83	22.6	6.62	7/16	92	88	83	79	74	70	65	61	56	52	47	43	38	..	..	..	..
3.94	1.22	1.84	25.6	7.50	1/2	105	99	94	89	84	79	74	68	63	58	53	48	43	..	..	..	..
						5″×3″ Angles																
1.50	.85	2.47	16.4	4.80	5/16	63	58	53	48	44	39	34	29	..	..	..	..	..	..	..	..	..
1.78	.84	2.48	19.6	5.72	3/8	74	69	63	57	51	46	40	34	..	..	..	..	..	..	..	..	..
2.04	.84	2.49	22.6	6.62	7/16	86	79	73	66	60	53	46	40	..	..	..	..	..	..	..	..	..
2.30	.83	2.50	25.6	7.50	1/2	97	90	82	74	67	59	52	44	..	..	..	..	..	..	..	..	..
						5″×3½″ Angles																
2.04	1.03	2.40	17.4	5.12	5/16	69	65	61	57	53	48	44	40	36	32	..	..	..	..	..	..	..
2.42	1.02	2.40	20.8	6.10	3/8	83	78	73	68	62	57	52	47	42	37	..	..	..	..	..	..	..
2.78	1.01	2.41	24.0	7.06	7/16	95	89	84	78	72	66	60	54	48	43	..	..	..	..	..	..	..
3.12	1.01	2.43	27.2	8.00	1/2	108	101	95	88	81	75	68	61	55	48	..	..	..	..	..	..	..
3.46	1.00	2.44	30.4	8.94	9/16	121	113	105	98	90	83	75	68	60	53	..	..	..	..	..	..	..
3.80	.99	2.45	33.6	9.84	5/8	132	124	116	107	99	91	82	74	66	57	..	..	..	..	..	..	..
4.12	.98	2.46	36.6	10.74	11/16	144	135	126	117	107	98	89	80	71	61	..	..	..	..	..	..	..
4.44	.98	2.48	39.6	11.62	3/4	156	146	136	126	116	106	96	86	76	66	..	..	..	..	..	..	..
						5″×5″ Angles																
4.84	1.56	2.22	24.6	7.22	3/8	104	100	96	92	88	84	81	77	73	69	65	61	57	53	49	46	42
5.58	1.55	2.23	28.6	8.36	7/16	120	116	111	107	102	98	93	88	84	79	75	70	66	61	57	52	48
6.30	1.54	2.24	32.4	9.50	1/2	136	131	126	121	116	111	105	100	95	90	85	79	74	69	64	59	54

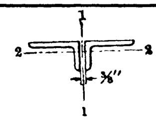
TABLE 44.—*Continued*

SAFE LOADS OF TWO ANGLE STRUTS, AXIS 2–2

EQUAL LEG, AND UNEQUAL LEG WITH LONG LEG TURNED OUT

AMERICAN BRIDGE COMPANY STANDARDS

Safe loads in thousands of pounds with respect to axis 2–2
$p = 16{,}000 - 70\ l/r$



To left of heavy line values of l/r do not exceed 125
To right of heavy line values of l/r do not exceed 150

Section Modulus S_2	Radius of Gyration r_2	Radius of Gyration r_1	Weight of Two Angles per Foot	Area of Two Angles	Thickness	Length in Feet 3	4	5	6	7	8	9	10	11	12	13	14	16	18	19	20	22	23
In.³	In.	In.	Lb.	In.²	In.																		
6″×3½″ Angles																							
2.46	.99	2.95	23.4	6.84	3/8	92	86	80	75	69	63	57	51	46	40	...	...	...	...	...	...	...	...
2.82	.98	2.96	27.0	7.94	7/16	107	100	93	86	79	73	66	59	52	45	...	...	...	...	...	...	...	...
3.18	.97	2.98	30.6	9.00	1/2	121	113	105	97	89	82	74	66	58	51	...	...	...	...	...	...	...	...
3.88	.96	3.00	37.8	11.10	5/8	148	139	129	119	110	100	90	80	71	61	...	...	...	...	...	...	...	...
6″×4″ Angles																							
3.20	1.17	2.87	24.6	7.22	3/8	100	95	90	84	79	74	69	64	59	53	48	43	...	...	...	...	...	...
3.70	1.16	2.88	28.6	8.36	7/16	116	110	104	97	91	85	79	73	67	61	55	49	...	...	...	...	...	...
4.16	1.15	2.90	32.4	9.50	1/2	131	124	117	110	103	96	90	83	76	69	62	55	...	...	...	...	...	...
4.62	1.14	2.91	36.2	10.62	9/16	147	139	131	123	115	107	100	92	84	76	68	60	...	...	...	...	...	...
5.08	1.13	2.92	40.0	11.72	5/8	161	153	144	135	127	118	109	100	92	83	74	66	...	...	...	...	...	...
5.52	1.13	2.93	43.6	12.82	11/16	177	167	157	148	138	129	119	110	100	91	81	72	...	...	...	...	...	...
5.94	1.12	2.94	47.2	13.88	3/4	191	180	170	160	149	139	128	118	108	97	87	76	...	...	...	...	...	...
6″×6″ Angles																							
7.06	1.88	2.62	29.8	8.72	3/8	128	124	120	116	112	108	105	101	97	93	89	85	77	69	66	62	54	50
8.14	1.87	2.63	34.4	10 12	7/16	148	144	139	135	130	126	121	117	112	107	103	98	89	80	75	71	62	57
9.22	1.86	2.64	39.2	11.50	1/2	168	163	158	153	148	142	137	132	127	122	117	111	101	91	85	80	70	65
10.28	1.85	2.65	43.8	12.86	9/16	188	182	177	171	165	159	153	147	142	136	130	124	112	101	95	89	77	71
11.32	1.84	2.66	48.4	14.22	5/8	208	202	195	189	182	176	169	163	156	150	143	137	124	111	104	98	85	78
12.34	1.83	2.67	53.0	15.56	11/16	228	220	213	206	199	192	185	178	170	163	156	149	135	120	113	106	92	..
13.32	1.83	2.68	57.4	16.88	3/4	247	239	231	224	216	208	200	193	185	177	169	162	146	131	123	115	100	..

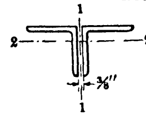
TABLE 45

Safe Loads of Two Angle Struts

Equal Leg, and Unequal Leg with Short Leg Turned Out

American Bridge Company Standards

Safe loads in thousands of pounds for least radius of gyration
$p = 16{,}000 - 70\ l/r$



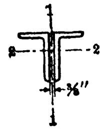
To left of heavy line values of l/r do not exceed 125
To right of heavy line values of l/r do not exceed 150

Section Modulus S_2	Radius of Gyration r_1	Radius of Gyration r_2	Weight of Two Angles per Foot	Area of Two Angles	Thickness	Length in Feet 3	4	5	6	7	8	9	10	11	12	13	14
In.^{3}	In.	In.	Lb.	In.^{2}	In.												
1½"×1½" Angles																	
.21	.78	.46	3.6	1.06	3/16	11	9	7	....	....	....	....	....	....	....	....	....
.27	.79	.45	4.8	1.38	1/4	14	12	9	....	....	....	....	....	....	....	....	....
2"×1½" Angles																	
.36	.67	.63	4.2	1.20	3/16	14	13	11	10	8	....	....	....	....	....	....	....
.46	.68	.63	5.4	1.56	1/4	19	17	15	12	10	....	....	....	....	....	....	....
1¾"×1¾" Angles																	
.28	.88	.54	4.4	1.24	3/16	14	12	10	8	....	....	....	....	....	....	....	....
.38	.89	.53	5.6	1.62	1/4	18	16	13	11	....	....	....	....	....	....	....	....
2"×2" Angles																	
.38	.98	.62	5.0	1.44	3/16	17	15	13	11	9	....	....	....	....	....	....	....
.50	.99	.61	6.4	1.88	1/4	22	20	17	15	12	....	....	....	....	....	....	....
2½"×2" Angles																	
.58	.92	.79	5.6	1.62	3/16	21	19	17	16	14	12	10	....	....	....	....	....
.76	.94	.78	7.4	2.12	1/4	27	25	23	20	18	16	13	....	....	....	....	....
.94	.95	.78	9.0	2.62	5/16	33	31	28	25	22	19	17	....	....	....	....	....
2½"×2½" Angles																	
.80	1.19	.77	8.2	2.38	1/4	30	28	25	22	20	17	15	....	....	....	....	....
.96	1.20	.76	10.0	2.94	5/16	37	34	31	28	24	21	18	....	....	....	....	....
1.14	1.21	.75	11.8	3.46	3/8	44	40	36	32	28	24	21	....	....	....	....	....
3"×2" Angles																	
1.08	.89	.95	8.2	2.38	1/4	31	29	27	25	22	20	18	16	13	....	....	....
1.32	.90	.95	10.0	2.94	5/16	39	36	33	31	28	25	22	20	17	....	....	....
1.56	.91	.94	11.8	3.46	3/8	46	43	39	36	33	30	27	23	20	....	....	....
3"×2½" Angles																	
1.12	1.13	.95	9.0	2.62	1/4	35	33	30	28	26	23	21	19	16	....	....	....
1.38	1.14	.94	11.2	3.24	5/16	43	40	37	34	32	29	26	23	20	....	....	....
1.62	1.16	.93	13.2	3.84	3/8	51	48	44	41	37	34	30	27	23	....	....	....
3"×3" Angles																	
1.16	1.39	.93	9.8	2.88	1/4	38	36	33	30	28	25	23	20	17	....	....	....
1.42	1.40	.92	12.2	3.56	5/16	47	44	41	37	34	31	28	24	21	....	....	....
1.66	1.41	.91	14.4	4.22	3/8	56	52	48	44	40	36	32	29	25	....	....	....
1.90	1.42	.91	16.6	4.86	7/16	64	60	55	51	46	42	37	33	28	....	....	....
2.14	1.44	.90	18.8	5.50	1/2	73	67	62	57	52	47	42	37	32	....	....	....

TABLE 45.—*Continued*

Safe Loads of Two Angle Struts
Short Leg Turned Out
American Bridge Company Standards

Safe loads in thousands of pounds for least radius of gyration
$p = 16{,}000 - 70\ l/r$

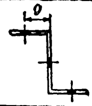


To left of heavy line values of l/r do not exceed 125
To right of heavy line values of l/r do not exceed 150

Section Modulus	Radius of Gyration		Weight of Two Angles per Foot	Area of Two Angles	Thickness	Length in Feet																		
S_2	r_1	r_2																						
In.³	In.	In.	Lb.	In.²	In.	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
4″×3″ Angles																								
2.46	1.30	1.27	14.4	4.18	5/16	59	56	53	50	48	45	42	39	37	34	31	28	25	...	...	...	...	...	...
2.92	1.31	1.26	17.0	4.96	3/8	69	66	63	59	56	53	50	46	43	40	36	33	30	...	...	...	...	...	...
3.36	1.32	1.25	19.6	5.74	7/16	80	76	73	69	65	61	57	53	49	46	42	38	34	...	...	...	...	...	...
3.78	1.33	1.25	22.2	6.50	1/2	91	87	82	78	73	69	65	60	56	52	47	43	38	...	...	...	...	...	...
4.18	1.34	1.24	24.8	7.24	9/16	101	96	91	86	82	77	72	67	62	57	52	47	42	...	...	...	...	...	...
4.60	1.36	1.23	27.2	7.96	5/8	111	106	100	95	89	84	78	73	68	62	57	51	46	...	...	...	...	...	...
5″×3″ Angles																								
3.78	1.22	1.61	16.4	4.80	5/16	67	64	60	57	54	50	47	44	40	37	34	31	27	...	...	...	...	...	...
4.48	1.23	1.61	19.6	5.72	3/8	80	76	72	68	64	60	56	52	49	45	41	37	33	...	...	...	...	...	...
5.16	1.24	1.60	22.6	6.62	7/16	92	88	83	79	75	70	66	61	57	52	48	43	39	...	...	...	...	...	...
5.82	1.25	1.59	25.6	7.50	1/2	105	100	95	90	85	80	75	70	65	60	54	49	44	...	...	...	...	...	...
6.46	1.26	1.58	28.6	8.36	9/16	117	111	106	100	95	89	84	78	72	67	61	56	50	...	...	...	...	...	...
7.10	1.28	1.57	31.4	9.22	5/8	129	123	117	111	105	99	93	87	81	75	69	63	57	51	...	...	...	...	...
5″×3½″ Angles																								
3.88	1.45	1.61	17.4	5.12	5/16	73	70	67	64	61	58	55	52	49	46	43	40	37	34	31	28	...	...	...
4.58	1.46	1.60	20.8	6.10	3/8	87	84	80	77	73	70	66	62	59	55	52	48	45	41	38	34	...	...	...
5.28	1.47	1.59	24.0	7.06	7/16	101	97	93	89	85	81	77	73	69	65	61	57	53	48	44	40	...	...	...
5.98	1.49	1.58	27.2	8.00	1/2	114	110	105	101	96	92	87	83	78	74	69	65	60	56	51	47	...	...	...
6.64	1.50	1.57	30.4	8.94	9/16	128	123	118	113	108	103	98	93	88	83	78	73	68	63	58	53	...	...	...
7.30	1.51	1.56	33.6	9.84	5/8	141	136	130	125	119	114	108	103	97	92	86	81	75	70	64	59	...	...	...
7.94	1.52	1.56	36.6	10.74	11/16	154	148	142	136	130	124	118	113	107	101	95	89	83	77	71	65	59	...	...
8.56	1.53	1.55	39.6	11.62	3/4	167	160	154	148	141	135	128	122	116	109	103	97	90	84	77	71	65	...	...
6″×3½″ Angles																								
6.50	1.39	1.94	23.4	6.84	3/8	97	93	89	85	81	76	72	68	64	60	56	52	47	43	39	...	...	...	...
7.50	1.40	1.93	27.0	7.94	7/16	113	108	103	98	94	89	84	79	74	70	65	60	55	51	46	...	...	...	...
8.48	1.41	1.92	30.6	9.00	1/2	128	123	117	112	106	101	96	90	85	80	74	69	64	58	53	...	...	...	...
9.44	1.42	1.91	34.2	10.06	9/16	143	137	131	125	119	113	107	102	96	90	84	78	72	66	60	...	...	...	...
10.38	1.43	1.90	37.8	11.10	5/8	158	152	145	138	132	125	119	112	106	99	93	86	80	73	67	...	...	...	...
11.30	1.45	1.89	41.2	12.12	11/16	173	166	159	152	145	138	131	124	117	110	103	96	89	82	75	68	...	...	...
12.20	1.46	1.89	44.8	13.12	3/4	187	180	172	165	157	150	142	135	127	119	112	104	97	89	82	74	...	...	...
6″×4″ Angles																								
6.64	1.62	1.93	24.6	7.22	3/8	104	101	97	93	89	86	82	78	74	71	67	63	59	56	52	48	45	41	..
7.66	1.63	1.92	28.6	8.36	7/16	121	117	112	108	104	99	95	91	86	82	78	74	69	65	61	56	52	48	..
8.66	1.65	1.91	32.4	9.50	1/2	138	133	128	123	118	113	109	104	99	94	89	84	79	75	70	65	60	55	..
9.66	1.66	1.90	36.2	10.62	9/16	154	148	143	138	132	127	122	116	111	105	100	95	89	84	79	73	68	62	..
10.62	1.67	1.90	40.0	11.72	5/8	170	164	158	152	146	140	134	129	123	117	111	105	99	93	87	81	76	70	.
11.56	1.68	1.89	43.6	12.82	11/16	186	179	173	167	160	154	147	141	135	128	122	115	109	103	96	90	83	77	71
12.50	1.70	1.88	47.2	13.88	3/4	202	195	188	181	174	167	160	153	147	140	133	126	119	112	105	99	92	85	78

TABLE 46

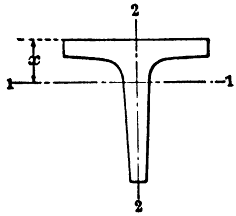
Properties and Elements of Z Bars



Nominal Size	Thickness	Actual Size: Web	Actual Size: Flange	Weight Per Foot	Area	Moments of Inertia, I (Inches⁴): Neutral Axis Through Center of Gravity Perpendicular to Web	Moments of Inertia, I (Inches⁴): Neutral Axis Through Center of Gravity Coincident with Web	Radii of Gyration, r (Inches): Neutral Axis Through Center of Gravity Perpendicular to Web	Radii of Gyration, r (Inches): Neutral Axis Through Center of Gravity Coincident with Web	Radii of Gyration, r (Inches): Least Radius, Neutral Axis Diagonal	Gage g	Max. Rivet or Bolt in Flange	Nominal Size
In.	In.	In.	In.	Lb.	Sq. In.						In.	In	In.
	3/8	6	3 1/2	15.6	4.59	25.32	9.11	2.35	1.41	0.83			
	7/16	6 1/16	3 9/16	18.3	5.39	29.80	10.95	2.35	1.43	0.83	2 1/4	7/8	
	1/2	6 1/8	3 5/8	21.0	6.19	34.36	12.87	2.36	1.44	0.84			
	9/16	6	3 1/2	22.7	6.68	34.64	12.59	2.28	1.37	0.81			
6	5/8	6 1/16	3 9/16	25.4	7.46	38.86	14.42	2.28	1.39	0.82	2 1/4	7/8	6
	11/16	6 1/8	3 5/8	28.0	8.25	43.18	16.34	2.29	1.41	0.84			
	3/4	6	3 1/2	29.3	8.63	42.12	15.44	2.21	1.34	0.81			
	13/16	6 1/16	3 9/16	31.9	9.40	46.13	17.27	2.22	1.36	0.82	2 1/4	7/8	
	7/8	6 1/8	3 5/8	34.6	10.17	50.22	19.18	2.22	1.37	0.83			
	5/16	5	3 1/4	11.6	3.40	13.36	6.18	1.98	1.35	0.75			
	3/8	5 1/16	3 5/16	13.9	4.10	16.18	7.65	1.99	1.37	0.76	2 1/8	7/8	
	7/16	5 1/8	3 3/8	16.4	4.81	19.07	9.20	1.99	1.38	0.77			
	1/2	5	3 1/4	17.9	5.25	19.19	9.05	1.91	1.31	0.74			
5	9/16	5 1/16	3 5/16	20.2	5.94	21.83	10.51	1.91	1.33	0.75	2 1/8	7/8	5
	5/8	5 1/8	3 3/8	22.6	6.64	24.53	12.06	1.92	1.35	0.76			
	11/16	5	3 1/4	23.7	6.96	23.68	11.37	1.84	1.28	0.73			
	3/4	5 1/16	3 5/16	26.0	7.64	26.16	12.83	1.85	1.30	0.74	2 1/8	7/8	
	13/16	5 1/8	3 3/8	28.3	8.33	28.70	14.36	1.86	1.31	0.76			
	1/4	4	3 1/16	8.2	2.41	6.28	4.23	1.62	1.33	0.67			
	5/16	4 1/16	3 1/8	10.3	3.03	7.94	5.46	1.62	1.34	0.68	2	3/4	
	3/8	4 1/8	3 3/16	12.4	3.66	9.63	6.77	1.62	1.36	0.69			
	7/16	4	3 1/16	13.8	4.05	9.66	6.73	1.55	1.29	0.66			
4	1/2	4 1/16	3 1/8	15.8	4.66	11.18	7.96	1.55	1.31	0.67	2	3/4	4
	9/16	4 1/8	3 3/16	17.9	5.27	12.74	9.26	1.55	1.33	0.68			
	5/8	4	3 1/16	18.9	5.55	12.11	8.73	1.48	1.25	0.66			
	11/16	4 1/16	3 1/8	20.9	6.14	13.52	9.95	1.48	1.27	0.67	2	3/4	
	3/4	4 1/8	3 3/16	23.0	6.75	14.07	11.24	1.49	1.29	0.68			
	1/4	3	2 11/16	6.7	1.97	2.87	2.81	1.21	1.19	0.55	1 5/8	3/4	
	5/16	3 1/16	2 3/4	8.4	2.48	3.64	3.64	1.21	1.21	0.56			
3	3/8	3	2 11/16	9.7	2.86	3.85	3.92	1.16	1.17	0.54	1 5/8	3/4	3
	7/16	3 1/16	2 3/4	11.4	3.36	4.57	4.75	1.17	1.19	0.55			
	1/2	3	2 11/16	12.5	3.69	4.59	4.85	1.12	1.15	0.53	1 5/8	3/4	
	9/16	3 1/16	2 3/4	14.2	4.18	5.26	5.70	1.12	1.17	0.54			

TABLE 47.

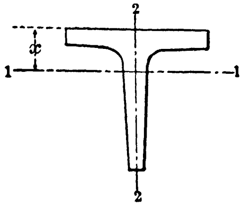
Elements of Carnegie Equal Tees.



Size.				Weight per Foot.	Area of Section.	Axis 1-1.				Axis 2-2.		
Flange.	Stem.	Min. Thickness. Flange.	Min. Thickness. Stem.			I	r	S	x	I	r	S
In.	In.	In.	In.	Lb.	$In.^2$	$In.^4$	In.	$In.^3$	In.	$In.^4$	In.	$In.^3$
4	4	$\frac{1}{2}$	$\frac{1}{2}$	13.5	3.97	5.7	1.20	2.0	1.18	2.8	0.84	1.4
4	4	$\frac{3}{8}$	$\frac{3}{8}$	10.5	3.09	4.5	1.21	1.6	1.13	2.1	0.83	1.1
$3\frac{1}{2}$	$3\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	11.7	3.44	3.7	1.04	1.5	1.05	1.9	0.74	1.1
$3\frac{1}{2}$	$3\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{8}$	9.2	2.68	3.0	1.05	1.2	1.01	1.4	0.73	0.81
3	3	$\frac{1}{2}$	$\frac{1}{2}$	9.9	2.91	2.3	0.88	1.1	0.93	1.2	0.64	0.80
3	3	$\frac{7}{16}$	$\frac{7}{16}$	8.9	2.59	2.1	0.89	0.98	0.91	1.0	0.63	0.70
3	3	$\frac{3}{8}$	$\frac{3}{8}$	7.8	2.27	1.8	0.90	0.86	0.88	0.90	0.63	0.60
3	3	$\frac{5}{16}$	$\frac{5}{16}$	6.7	1.95	1.6	0.90	0.74	0.86	0.75	0.62	0.50
$2\frac{1}{2}$	$2\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{8}$	6.4	1.87	1.0	0.74	0.59	0.76	0.52	0.53	0.42
$2\frac{1}{2}$	$2\frac{1}{2}$	$\frac{5}{16}$	$\frac{5}{16}$	5.5	1.60	0.88	0.74	0.50	0.74	0.44	0.52	0.35
$2\frac{1}{4}$	$2\frac{1}{4}$	$\frac{5}{16}$	$\frac{5}{16}$	4.9	1.43	0.65	0.67	0.41	0.68	0.33	0.48	0.29
$2\frac{1}{4}$	$2\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	4.1	1.19	0.52	0.66	0.32	0.65	0.25	0.46	0.22
2	2	$\frac{5}{16}$	$\frac{5}{16}$	4.3	1.26	0.44	0.59	0.31	0.61	0.23	0.43	0.23
2	2	$\frac{1}{4}$	$\frac{1}{4}$	3.56	1.05	0.37	0.59	0.26	0.59	0.18	0.42	0.18
$1\frac{3}{4}$	$1\frac{3}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	3.09	0.91	0.23	0.51	0.19	0.54	0.12	0.37	0.14
$1\frac{1}{2}$	$1\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{4}$	2.47	0.73	0.15	0.45	0.14	0.47	0.08	0.32	0.10
$1\frac{1}{2}$	$1\frac{1}{2}$	$\frac{3}{16}$	$\frac{3}{16}$	1.94	0.57	0.11	0.45	0.11	0.44	0.06	0.32	0.08
$1\frac{1}{4}$	$1\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	2.02	0.59	0.08	0.37	0.10	0.40	0.05	0.28	0.07
$1\frac{1}{4}$	$1\frac{1}{4}$	$\frac{3}{16}$	$\frac{3}{16}$	1.59	0.47	0.06	0.37	0.07	0.38	0.03	0.27	0.05
1	1	$\frac{3}{16}$	$\frac{3}{16}$	1.25	0.37	0.03	0.29	0.05	0.32	0.02	0.22	0.04
1	1	$\frac{1}{8}$	$\frac{1}{8}$	0.89	0.26	0.02	0.30	0.03	0.29	0.01	0.21	0.02

TABLE 48.

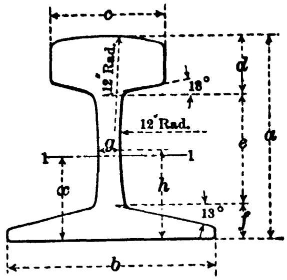
Elements of Carnegie Unequal Tees.



Section Index.	Size.				Weight per Foot.	Area of Section.	Axis 1-1.				Axis 2-2.		
	Flange.	Stem.	Minimum Thickness. Flange.	Minimum Thickness. Stem.			I	r	S	x	I	r	S
	In.	In.	In.	In.	Lb.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.	In.³
T 50	5	3	$\frac{1}{2}$	$\frac{13}{32}$	13.4	3.93	2.4	0.78	1.1	0.73	5.4	1.17	2.2
T 51	5	$2\frac{1}{2}$	$\frac{3}{8}$	$\frac{7}{16}$	10.9	3.18	1.5	0.68	0.78	0.63	4.1	1.14	1.6
T 52	$4\frac{1}{2}$	$3\frac{1}{2}$	$\frac{7}{16}$	$\frac{11}{16}$	15.7	4.60	5.1	1.05	2.1	1.11	3.7	0.90	1.7
T 54	$4\frac{1}{2}$	3	$\frac{3}{8}$	$\frac{3}{8}$	9.8	2.88	2.1	0.84	0.91	0.74	3.0	1.02	1.3
T 53	$4\frac{1}{2}$	3	$\frac{5}{16}$	$\frac{5}{16}$	8.4	2.46	1.8	0.85	0.78	0.71	2.5	1.01	1.1
T 56	$4\frac{1}{2}$	$2\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{8}$	9.2	2.68	1.2	0.67	0.63	0.59	3.0	1.05	1.3
T 55	$4\frac{1}{2}$	$2\frac{1}{2}$	$\frac{5}{16}$	$\frac{5}{16}$	7.8	2.29	1.0	0.68	0.54	0.57	2.5	1.05	1.1
T 57	4	5	$\frac{1}{2}$	$\frac{1}{2}$	15.3	4.50	10.8	1.55	3.1	1.56	2.8	0.79	1.4
T 58	4	5	$\frac{3}{8}$	$\frac{3}{8}$	11.9	3.49	8.5	1.56	2.4	1.51	2.1	0.78	1.1
T 59	4	$4\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	14.4	4.23	7.9	1.37	2.5	1.37	2.8	0.81	1.4
T 60	4	$4\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{8}$	11.2	3.29	6.3	1.39	2.0	1.31	2.1	0.80	1.1
T 61	4	3	$\frac{3}{8}$	$\frac{3}{8}$	9.2	2.68	2.0	0.86	0.90	0.78	2.1	0.89	1.1
T 44	4	3	$\frac{5}{16}$	$\frac{5}{16}$	7.8	2.29	1.7	0.87	0.77	0.75	1.8	0.88	0.88
T 62	4	$2\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{8}$	8.5	2.48	1.2	0.69	0.62	0.62	2.1	0.92	1.0
T 63	4	$2\frac{1}{2}$	$\frac{5}{16}$	$\frac{5}{16}$	7.2	2.12	1.0	0.69	0.53	0.60	1.8	0.91	0.88
T 64	4	2	$\frac{3}{8}$	$\frac{3}{8}$	7.8	2.27	0.60	0.52	0.40	0.48	2.1	0.96	1.1
T 65	4	2	$\frac{5}{16}$	$\frac{5}{16}$	6.7	1.95	0.53	0.52	0.34	0.46	1.8	0.95	0.88
T 66	$3\frac{1}{2}$	4	$\frac{1}{2}$	$\frac{1}{2}$	12.6	3.70	5.5	1.21	2.0	1.24	1.9	0.72	1.1
T 67	$3\frac{1}{2}$	4	$\frac{3}{8}$	$\frac{3}{8}$	9.8	2.88	4.3	1.23	1.5	1.19	1.4	0.70	0.81
T 69	$3\frac{1}{2}$	3	$\frac{1}{2}$	$\frac{1}{2}$	10.8	3.17	2.4	0.87	1.1	0.88	1.9	0.77	1.1
T 70	$3\frac{1}{2}$	3	$\frac{3}{8}$	$\frac{3}{8}$	8.5	2.48	1.9	0.88	0.89	0.83	1.4	0.75	0.81
T 71	$3\frac{1}{2}$	3	$\frac{5}{16}$	$\frac{3}{8}$	7.5	2.20	1.8	0.91	0.85	0.85	1.2	0.74	0.68
T 72	3	4	$\frac{1}{2}$	$\frac{1}{2}$	11.7	3.44	5.2	1.23	1.9	1.32	1.2	0.59	0.81
T 73	3	4	$\frac{7}{16}$	$\frac{7}{16}$	10.5	3.06	4.7	1.23	1.7	1.29	1.1	0.59	0.70
T 74	3	4	$\frac{3}{8}$	$\frac{3}{8}$	9.2	2.68	4.1	1.24	1.5	1.27	0.90	0.58	0.60
T 75	3	$3\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	10.8	3.17	3.5	1.06	1.5	1.12	1.2	0.62	0.80
T 76	3	$3\frac{1}{2}$	$\frac{7}{16}$	$\frac{7}{16}$	9.7	2.83	3.2	1.06	1.3	1.10	1.0	0.60	0.69
T 77	3	$3\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{8}$	8.5	2.48	2.8	1.07	1.2	1.07	0.93	0.61	0.62
T 78	3	$2\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{8}$	7.1	2.07	1.1	0.72	0.60	0.71	0.89	0.66	0.59
T 79	3	$2\frac{1}{2}$	$\frac{5}{16}$	$\frac{5}{16}$	6.1	1.77	0.94	0.73	0.52	0.68	0.75	0.65	0.50
T 31	3	$2\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{4}$	5.0	1.47	0.78	0.73	0.43	0.66	0.61	0.64	0.40
T 82	$2\frac{1}{2}$	3	$\frac{3}{8}$	$\frac{3}{8}$	7.1	2.07	1.7	0.91	0.84	0.95	0.53	0.51	0.42
T 83	$2\frac{1}{2}$	3	$\frac{5}{16}$	$\frac{5}{16}$	6.1	1.77	1.5	0.92	0.72	0.92	0.44	0.50	0.35
T 86	$2\frac{1}{2}$	$1\frac{1}{4}$	$\frac{3}{16}$	$\frac{3}{16}$	2.87	0.84	0.08	0.31	0.09	0.32	0.29	0.58	0.23
T 87	2	$1\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{4}$	3.09	0.91	0.16	0.42	0.15	0.42	0.18	0.45	0.18
T 519	$1\frac{1}{2}$	2	$\frac{3}{16}$	$\frac{3}{16}$	2.45	0.72	0.27	0.61	0.19	0.63	0.06	0.92	0.08
T 605	$1\frac{1}{2}$	$1\frac{1}{4}$	$\frac{1}{8}$	$\frac{1}{8}$	1.25	0.37	0.05	0.37	0.05	0.33	0.04	0.32	0.05
T 603	$1\frac{1}{4}$	$\frac{5}{8}$	No. 9	$\frac{1}{8}$	0.88	0.26	0.01	0.16	0.01	0.16	0.02	0.31	0.04

TABLE 49.

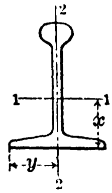
Elements of A. S. C. E. and Light Rails.



Section Index.	Weight per Yard.	Area of Section.	Dimensions.								Axis 1-1.			
			a	b	c	d	e	f	g	h	I	r	S	x
	Pounds.	In.²	In.	In.	In.	In.	In.	In.	In.	In.	In.⁴	In.	In.³	In.
110A	110	10.80	6 1/8	6 1/8	2 7/8	1 25/32	3 11/32	1	37/64	2 43/64	55.2	2.26	17.2	2.92
100A	100	9.84	5 3/4	5 3/4	2 3/4	1 45/64	3 5/64	31/32	9/16	2 65/128	44.0	2.11	14.6	2.73
95A	95	9.28	5 9/16	5 9/16	2 11/16	1 41/64	2 63/64	15/16	9/16	2 55/128	38.8	2.05	13.3	2.65
90A	90	8.83	5 3/8	5 3/8	2 5/8	1 19/32	2 55/64	59/64	9/16	2 45/128	34.4	1.97	12.2	2.55
85A	85	8.33	5 3/16	5 3/16	2 9/16	1 35/64	2 3/4	57/64	9/16	2 17/64	30.1	1.90	11.1	2.47
80A	80	7.86	5	5	2 1/2	1 1/2	2 5/8	7/8	35/64	2 3/16	26.4	1.83	10.1	2.38
75A	75	7.33	4 13/16	4 13/16	2 15/32	1 27/64	2 35/64	27/32	17/32	2 15/128	22.9	1.77	9.1	2.30
70A	70	6.81	4 5/8	4 5/8	2 7/16	1 11/32	2 15/32	13/16	33/64	2 3/64	19.7	1.70	8.2	2.22
65A	65	6.33	4 7/16	4 7/16	2 13/32	1 9/32	2 3/8	25/32	1/2	1 31/32	16.9	1.63	7.4	2.14
60A	60	5.93	4 1/4	4 1/4	2 3/8	1 7/32	2 17/64	49/64	31/64	1 115/128	14.6	1.57	6.6	2.05
55A	55	5.38	4 1/16	4 1/16	2 1/4	1 11/64	2 11/64	23/32	15/32	1 108/128	12.0	1.50	5.7	1.97
50A	50	4.87	3 7/8	3 7/8	2 1/8	1 1/8	2 1/16	11/16	7/16	1 23/32	9.9	1.43	5.0	1.88
45A	45	4.40	3 11/16	3 11/16	2	1 1/16	1 31/32	21/32	27/64	1 41/64	8.1	1.36	4.3	1.78
40A	40	3.94	3 1/2	3 1/2	1 7/8	1 1/64	1 55/64	5/8	25/64	1 71/128	6.6	1.29	3.6	1.68
35A	35	3.44	3 5/16	3 5/16	1 3/4	61/64	1 25/32	37/64	23/64	1 15/32	5.2	1.23	3.0	1.60
30A	30	3.00	3 1/8	3 1/8	1 11/16	7/8	1 23/32	17/32	21/64	1 25/64	4.1	1.16	2.5	1.52
25A	25	2.39	2 3/4	2 3/4	1 1/2	25/32	1 31/64	31/64	19/64	1 29/128	2.5	1.02	1.8	1.33
20A	20	2.00	2 5/8	2 5/8	1 11/32	23/32	1 15/32	7/16	1/4	1 11/64	1.9	0.99	1.4	1.27
16A	16	1.55	2 3/8	2 3/8	1 11/64	41/64	1 23/64	3/8	7/32	1 7/128	1.2	0.89	1.0	1.15
14A	14	1.34	2 1/16	2 1/16	1 1/16	5/8	1 3/32	11/32	1/4	57/64	0.76	0.75	0.73	1.02
12A	12	1.18	2	2	1	9/16	1 3/32	11/32	3/16	57/64	0.66	0.75	0.63	0.96
10A	10	0.96	1 3/4	1 3/4	15/16	33/64	15/16	19/64	3/16	49/64	0.40	0.65	0.46	0.87
8A	8	0.77	1 9/16	1 9/16	13/16	15/32	13/16	9/32	5/32	11/16	0.26	0.58	0.32	0.75

TABLE 50.

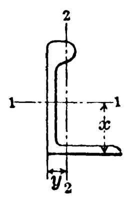
Elements of Carnegie Bulb Beams.



Depth of Beam.	Wt. per Foot.	Area of Section.	Width of Flange.	Thickness of Web.	Axis 1-1.				Axis 2-2.			
					I	r	S	x	I	r	S	y
In.	Lb.	In.^{2}	In.	In.	In.^{4}	In.	In.^{3}	In.	In.^{4}	In.	In.^{3}	In.
10	36.6	10.62	5.500	0.625	140.4	3.64	25.3	4.45	7.6	0.84	2.8	2.75
10	28.1	8.12	5.250	0.375	118.6	3.82	20.7	4.28	6.3	0.88	2.4	2.63
9	30.1	8.83	5.125	0.563	95.8	3.29	19.4	4.06	5.4	0.78	2.1	2.56
9	24.3	7.15	4.938	0.375	84.0	3.43	16.6	3.95	4.6	0.80	1.9	2.47
8	24.2	7.11	5.156	0.469	62.8	2.97	14.1	3.54	4.5	0.79	1.7	2.58
8	20.0	5.86	5.000	0.313	55.6	3.08	12.2	3.43	3.9	0.82	1.6	2.50
7	23.3	6.85	5.094	0.531	45.5	2.57	11.7	3.11	4.3	0.79	1.7	2.55
7	18.1	5.32	4.875	0.313	38.8	2.70	9.7	2.98	3.6	0.82	1.5	2.44
6	17.2	5.00	4.524	0.430	24.4	2.20	7.2	2.61	2.7	0.73	1.2	2.26
6	14.0	4.11	4.375	0.281	21.6	2.28	6.1	2.46	2.2	0.72	1.0	2.19

TABLE 51.

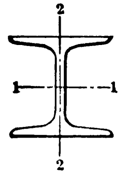
Elements of Carnegie Bulb Angles.



Depth of Beam.	Wt. per Foot.	Area of Section.	Width of Flange.	Thickness of Web.	Axis 1-1.				Axis 2-2.			
					I	r	S	x	I	r	S	y
In.	Lb.	In.^{2}	In.	In.	In.^{4}	In.	In.^{3}	In.	In.^{4}	In.	In.^{3}	In.
10	32.0	9.41	3.500	0.625	116.0	3.51	21.6	4.62	6.2	0.82	2.3	0.77
10	26.6	7.80	3.500	0.484	104.2	3.66	19.9	4.75	5.0	0.80	1.8	0.72
9	21.8	6.41	3.500	0.438	69.3	3.33	14.5	4.21	4.3	0.82	1.5	0.72
8	19.3	5.66	3.500	0.406	48.8	2.95	11.7	3.83	3.7	0.81	1.3	0.71
7	20.0	5.81	3.000	0.500	36.6	2.51	10.0	3.34	2.9	0.71	1.3	0.70
7	18.3	5.37	3.000	0.438	34.9	2.56	9.6	3.36	2.6	0.69	1.1	0.68
7	16.1	4.71	3.000	0.344	32.2	2.61	8.7	3.30	2.7	0.76	1.2	0.72
6	17.3	5.06	3.000	0.500	23.9	2.16	7.6	2.84	2.5	0.70	1.1	0.71
6	15.0	4.38	3.000	0.406	21.1	2.19	6.7	2.84	2.3	0.72	1.0	0.69
6	13.8	4.04	3.000	0.375	20.1	2.21	6.6	2.96	1.9	0.69	0.82	0.65
6	12.4	3.62	3.000	0.313	18.6	2.28	5.7	2.71	1.8	0.70	0.75	0.64
5	10.0	2.94	2.500	0.313	10.2	1.86	4.1	2.49	0.95	0.57	0.49	0.57
4	14.3	4.21	3.500	0.500	8.7	1.44	3.7	1.65	3.9	0.96	1.5	0.99
4	11.9	3.48	3.500	0.375	7.0	1.50	3.5	1.77	3.1	0.94	1.2	0.94

TABLE 52.

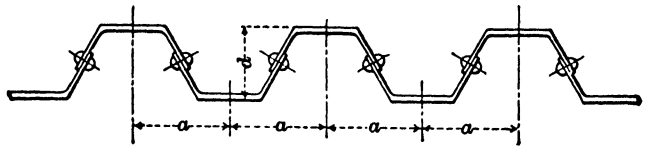
Elements of Carnegie H Beams.



Depth of Beam.	Wt. per Foot.	Area of Section.	Width of Flange.	Thickness of Web.	Axis 1-1.			Axis 2-2.		
					I	r	S	I	r	S
In.	Lb.	$In.^2$	In.	In.	$In.^4$	In.	$In.^3$	$In.^4$	In.	$In.^3$
8	34.0	10.00	8.0	.375	115.4	3.40	28.9	35.1	1.87	8.8
6	23.8	7.00	6.0	.313	45.1	2.54	15.0	14.7	1.45	4.9
5	18.7	5.50	5.0	.313	23.8	2.08	9.5	7.9	1.20	3.1
4	13.6	4.00	4.0	.313	10.7	1.63	5.3	3.6	0.95	1.8

TABLE 53.

Carnegie Trough Plates.



Elements of Trough Plates.

Single Section.			Riveted Section.			
Section Index.	Size, Inches.	Weight per Foot, Pounds.	a, Inches.	d, Inches.	Weight per Square Foot, Pounds.	Section Modulus, One Foot Width, $Inches^3$.
M 14	$9\frac{1}{2} \times 3\frac{3}{4}$	23.2	8	$6\frac{1}{2}$	34.8	15.58
M 13	$9\frac{1}{2} \times 3\frac{3}{4}$	21.4	8	$6\frac{3}{8}$	32.1	14.28
M 12	$9\frac{1}{2} \times 3\frac{3}{4}$	19.7	8	$6\frac{1}{4}$	29.6	13.00
M 11	$9\frac{1}{2} \times 3\frac{3}{4}$	18.0	8	$6\frac{1}{8}$	27.0	11.79
M 10	$9\frac{1}{2} \times 3\frac{3}{4}$	16.3	8	6	24.5	10.69

Allowable Uniform Load in Pounds per Square Foot.

Span in Feet.	Fiber Stress, 16,000 Lbs. per Sq. In.					Fiber Stress, 12,000 Lbs. per Sq. In.				
	M 14	M 13	M 12	M 11	M 10	M 14	M 13	M 12	M 11	M 10
5	6647	6093	5547	5030	4561	4986	4570	4160	3773	3421
6	4616	4231	3852	3493	3167	3462	3173	2889	2620	2376
7	3392	3109	2830	2567	2327	2543	2331	2124	1925	1745
8	2597	2380	2167	1965	1782	1948	1785	1625	1474	1336
9	2052	1880	1712	1553	1408	1539	1410	1284	1164	1058
10	1662	1523	1387	1258	1140	1246	1142	1040	943	855
11	1373	1259	1146	1039	942	1030	944	860	780	707
12	1154	1058	963	873	792	866	793	722	655	594
13	983	901	821	744	675	738	676	615	558	506
14	848	777	707	642	582	636	583	531	481	436
15	739	677	616	559	507	554	509	462	419	381
16	649	595	542	491	445	487	446	406	368	334
17	575	527	480	435	395	431	395	360	328	296
18	513	470	428	388	352	385	353	321	291	264
19	460	422	384	349	316	345	316	288	261	237
20	415	381	347	314	285	312	286	260	236	214

The values given in above tables are the safe loads per square foot of floor surface and are based upon the average resistance of the riveted portion within distance a.

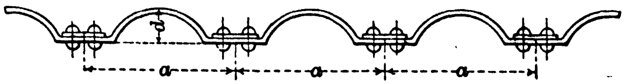
The weight of the plates are included in the safe loads and must be deducted to obtain the net superimposed safe load.

Safe loads for other fiber stresses than those given in table may be obtained from the values given by direct proportion of the fiber stresses.

The weight per square foot does not include the weight of rivet heads or other details.

TABLE 54.

Carnegie Corrugated Plates.



Elements of Corrugated Plates.

Single Section.			Riveted Section.			
Section Index.	Size, Inches.	Weight per Foot, Pounds.	a, Inches.	d, Inches.	Weight per Square Foot, Pounds.	Section Modulus, One-Foot Width, Inches³.
M 35	$12\frac{3}{16} \times 2\frac{7}{8}$	23.7	$12\frac{3}{16}$	$2\frac{7}{8}$	23.3	4.39
M 34	$12\frac{3}{16} \times 2\frac{13}{16}$	20.8	$12\frac{3}{16}$	$2\frac{13}{16}$	20.4	3.84
M 33	$12\frac{3}{16} \times 2\frac{3}{4}$	17.8	$12\frac{3}{16}$	$2\frac{3}{4}$	17.5	3.28
M 32	$8\frac{3}{4} \times 1\frac{5}{8}$	12.0	$8\frac{3}{4}$	$1\frac{5}{8}$	16.5	1.95
M 31	$8\frac{3}{4} \times 1\frac{9}{16}$	10.1	$8\frac{3}{4}$	$1\frac{9}{16}$	13.8	1.55
M 30	$8\frac{3}{4} \times 1\frac{1}{2}$	8.1	$8\frac{3}{4}$	$1\frac{1}{2}$	11.5	1.10

Allowable Uniform Load in Pounds per Square Foot.

Span in Feet.	Fiber Stress, 16,000 lb. per sq. in.						Fiber Stress, 12,000 lb. per sq. in.					
	M 35	M 34	M 33	M 32	M 31	M 30	M 35	M 34	M 33	M 32	M 31	M 30
5	1873	1638	1400	832	661	469	1405	1229	1050	624	496	352
6	1301	1138	972	578	459	326	976	853	729	433	344	244
7	956	836	714	425	337	240	717	627	536	318	253	180
8	732	640	547	325	258	183	549	480	410	244	194	138
9	578	506	432	257	204	145	434	379	324	193	153	109
10	468	410	350	208	165	117	351	307	262	156	124	88
11	387	339	289	172	137	97	290	255	217	129	103	73
12	325	284	243	144	115	82	244	213	182	108	86	61
13	277	242	207	123	98	69	208	182	155	92	73	52
14	239	209	179	106	84	60	179	157	134	80	63	45
15	208	182	156	92	74	52	156	137	117	69	51	39

The values given in above tables are the safe loads per square foot of floor surface and are based upon the average resistance of the riveted portion within distance a.

The weight of the plates are included in the safe loads and must be deducted to obtain the net superimposed safe load.

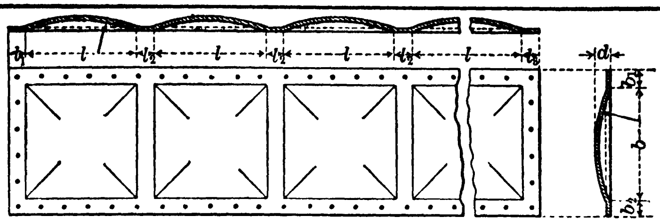
Safe loads for other fiber stresses than those given in table may be obtained from the values given by direct proportion of the fiber stresses.

The weight per square foot does not include the weight of splice bars, rivet heads or other details.

TABLE 55.

Buckle Plates.

AMERICAN BRIDGE COMPANY STANDARD.



Die Number.	Size of Buckle. Side l, Ft.-In.	Size of Buckle. Side b, Ft.-In.	Rise d, In.	Radii of Buckle. Side l, Ft.-In.	Radii of Buckle. Side b, Ft.-In.	Number of Buckles in One Plate.	Widths of Flanges and Fillets. End Flanges l_1, l_3.	Widths of Flanges and Fillets. Fillets l_2.	Widths of Flanges and Fillets. Side Flanges b_1, b_2.
1	3-11	4- 6	3½	6- 8⅝	8- 9⅞	1 to 8	Preferably made alike. Minimum = 2″——Maximum = 1′-6″. If wider than 1′-6″ use angles riveted across the plate for stiffeners	Minimum = 2″——Maximum = 6″. 4″ or less preferred	Preferably made alike. Minimum = 2″——Maximum = 6½″. *Note.*—When the side flanges b_1 and b_2 are of unequal width, the material should be ordered wide enough to make two flanges of the greater width, the narrower flange to be sheared to required width after buckling.
2	4- 6	3-11	3½	8- 9⅞	6- 8⅝	1 to 7			
3	3-11	3- 6	3	7- 9½	6- 3	1 to 8			
4	3- 6	3-11	3	6- 3	7- 9½	1 to 9			
5	3- 9	3- 9	3	7- 1⅞	7- 1⅞	1 to 8			
6	3- 1	3- 9	3	4-10⅝	7- 1⅞	1 to 10			
7	3- 9	3- 1	3	7- 1⅞	4-10⅝	1 to 8			
8	3- 8	3- 8	2	10- 2	10- 2	1 to 8			
9	2- 8	3- 8	2	5- 5	10- 2	1 to 11			
10	3- 8	2- 8	2	10- 2	5- 5	1 to 8			
11	2- 2	3- 8	2	3- 7¼	10- 2	1 to 14			
12	3- 8	2- 2	2	10- 2	3- 7¼	1 to 8			
13	3- 0	3- 0	2	6-10	6-10	1 to 10			
14	2- 9	2- 9	3	3-10⅞	3-10⅞	1 to 11			
19	2- 6	2- 9	2½	3-10¼	4- 7⅞	1 to 12			
20	2- 9	2- 6	2½	4- 7⅞	3-10¼	1 to 11			
21	2- 6	2- 6	2½	3-10¼	3-10¼	1 to 12			
22	3- 5	3- 6	3	5-11 9/16	6- 3	1 to 9			
23	3- 6	3- 5	3	6- 3	5-11 9/16	1 to 9			
24	3- 6	3- 9	3	6- 3	7- 1⅞	1 to 9			
25	3- 9	3- 6	3	7- 1⅞	6- 3	1 to 8			
26	3- 2	3- 1	3	5- 1 21/32	4-10⅝	1 to 9			
27	3- 1	3- 2	3	4-10⅝	5- 1 21/32	1 to 10			
28	3- 0	3- 1	3	4- 7½	4-10⅝	1 to 10			
29	3- 1	3- 0	3	4-10⅝	4- 7½	1 to 10			
30	2- 6	2- 0	2½	3-10¼	2- 6 1/16	1 to 12			
31	2- 0	2- 6	2½	2- 6 1/16	3-10¼	1 to 15			
32	5- 6	3- 6	3½	13- 1 21/64	5- 4¾	1 to 5			
33	3- 6	5- 6	3½	5- 4¾	13- 1 21/64	1 to 9			
34	4- 0	4- 0	3	8- 1½	8- 1½	1 to 7			

Plates are steel ¼″, 5/16″, ⅜″ or 7/16″ thick.

Plates of greater length than given in table may be made by splicing with bars, angles, or tees.

All plates are made with buckles up, unless otherwise ordered. When buckles are turned down, a drain hole should be punched in the center of each buckle and should be shown on sketch.

Buckles of different sizes should not be used as it increases the cost of the plate.

Connection holes are generally for ⅝″, ¾″ or ⅞″ rivets or bolts. Different sized holes in same plate will increase the cost of the plate.

Spacing for holes lengthwise of plate should be in multiples of 3″ and should not exceed 12″. Odd spaces to be at end of plate and in even ¼″. Minimum spacing crosswise 4½″, usually 6″.

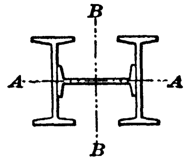
Die number must be shown on drawings.

Sketches for Buckle Plates should indicate allowable overrun in length and width.

TABLE 56.

Properties of Column Sections.

Properties of Three I-Beam Section.



Minimum I-Beam for Web.

Series I and II.		Series I.							Series II.						
Flange Beams.		Web Beam.		Total Area.	Moments of Inertia and Radii of Gyration.				Web Beam.		Total Area.	Moments of Inertia and Radii of Gyration.			
					Axis A–A.		Axis B–B.					Axis A–A.		Axis B–B.	
Depth.	Weight.	Depth.	Weight.		I_A	r_A	I_B	r_B	Depth.	Weight.		I_A	r_A	I_B	r_B
In.	Lb.	In.	Lb.	In.²	In.⁴	In.	In.⁴	In.	In.	Lb.	In.²	In.⁴	In.	In.⁴	In.
10	25	8	18	20.07	248	3.51	325	4.02	9	21	21.05	249	3.44	418	4.45
“	25	10	25	22.11	251	3.37	528	4.89	12	31.5	24.00	254	3.25	788	5.73
“	30	8	18	22.97	272	3.44	387	4.11	9	21	23.95	274	3.38	494	4.54
“	30	10	25	25.01	275	3.32	619	4.97	12	31.5	26.90	278	3.21	915	5.83
“	35	8	18	25.91	297	3.38	455	4.19	9	21	26.89	298	3.33	576	4.63
“	35	10	25	27.95	300	3.27	717	5.06	12	31.5	29.84	302	3.18	1050	5.93
12	31.5	10	25	25.89	439	4.12	635	4.95	12	31.5	27.78	441	3.98	941	5.82
“	31.5	15	42	31.00	446	3.79	1552	7.07	18	55	34.45	453	3.63	2373	8.30
“	35	10	25	27.95	464	4.07	703	5.01	12	31.5	29.84	466	3.95	1032	5.88
“	35	15	42	33.06	471	3.78	1688	7.14	18	55	36.51	478	3.62	2565	8.38
“	40	10	25	31.05	545	4.19	797	5.06	12	31.5	32.94	547	4.08	1162	5.94
“	40	15	42	36.16	552	3.91	1884	7.22	18	55	39.61	559	3.76	2841	8.47
15	42	10	25	32.33	890	5.24	828	5.06	12	31.5	34.22	893	5.11	1206	5.94
“	42	15	42	37.44	898	4.89	1953	7.22	18	55	40.89	905	4.70	2939	8.48
“	45	10	25	33.85	919	5.21	876	5.09	12	31.5	35.74	921	5.07	1274	5.97
“	45	15	42	38.96	926	4.87	2054	7.26	18	55	42.41	933	4.69	3082	8.53
“	50	10	25	36.79	974	5.14	974	5.14	12	31.5	38.68	976	5.02	1408	6.04
“	50	15	42	41.90	981	4.84	2254	7.33	18	55	45.35	988	4.67	3360	8.61
“	60	10	25	42.71	1225	5.42	1165	5.22	12	31.5	44.60	1228	5.24	1668	6.11
“	60	15	42	47.82	1233	5.07	2641	7.43	18	55	51.27	1239	4.91	3901	8.72
18	55	12	31.5	41.12	1601	6.24	1496	6.03	15	42	44.34	1606	6.02	2388	7.35
“	55	18	55	47.79	1612	5.81	3552	8.62	20	65	50.94	1619	5.64	4546	9.44
“	60	12	31.5	44.56	1693	6.16	1652	6.09	15	42	47.78	1698	5.96	2622	7.41
“	60	18	55	51.23	1705	5.77	3879	8.70	20	65	54.38	1712	5.61	4943	9.53
“	65	12	31.5	47.50	1773	6.09	1789	6.12	15	42	50.72	1778	5.92	2827	7.47
“	65	18	55	54.17	1784	5.74	4163	8.77	20	65	57.32	1791	5.59	5288	9.60
“	70	12	31.5	50.44	1852	6.06	1930	6.19	15	42	53.66	1857	5.88	3035	7.52
“	70	18	55	57.11	1864	5.71	4452	8.84	20	65	60.26	1871	5.57	5639	9.66
20	65	15	42	50.64	2354	6.82	2790	7.42	18	55	54.09	2360	6.60	4116	8.72
“	65	20	65	57.24	2367	6.43	5234	9.56	24	80	61.48	2382	6.23	7870	11.31
“	70	15	42	53.66	2454	6.76	2997	7.48	18	55	57.11	2461	6.56	4406	8.78
“	70	20	65	60.26	2468	6.40	5586	9.63	24	80	64.50	2483	6.21	8363	11.39
“	75	15	42	56.60	2552	6.71	3203	7.52	18	55	60.05	2559	6.53	4692	8.84
“	75	20	65	63.20	2566	6.37	5933	9.69	24	80	67.44	2581	6.19	8851	11.46
24	80	15	42	59.12	4190	8.42	3329	7.50	18	55	62.57	4197	8.18	4872	8.82
“	80	20	65	65.72	4204	8.00	6155	9.68	24	80	69.96	4219	7.76	9173	11.45
“	85	15	42	62.48	4352	8.35	3561	7.55	18	55	65.93	4358	8.13	5194	8.87
“	85	20	65	69.08	4365	7.95	6548	9.73	24	80	73.32	4380	7.73	9723	11.51
“	90	15	42	65.42	4493	8.29	3767	7.60	18	55	68.87	4499	8.08	5481	8.92
“	90	20	65	72.02	4506	7.91	68[illegible]3	9.78	24	80	76.26	4521	7.70	10207	11.56
“	100	15	42	71.28	4775	8.18	4187	7.66	18	55	74.73	4782	8.00	6060	9.01
“	100	20	65	77.88	4789	7.84	7597	9.88	24	80	82.12	4804	7.65	11193	11.66

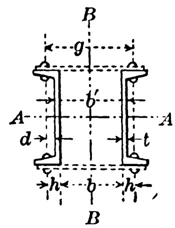
Heavier web beams, of same depth as those given in table, may be substituted by subtracting area and moments of inertia of given beam, respectively, from values given in table, and adding the corresponding properties of new beam. The radii of gyration must then be recalculated from the formula $r = \sqrt{I \div A}$.

Note 1924. This table was calculated in 1914. Minimum I-beams have been used for webs for which all values are as given in Table 7 except "weights." For minimum I-beams used for flanges, all properties given in this table except "weights" are exact. For other than minimum flanges check "total areas"; other values are correct within four tenths of one per cent.

TABLE 57.

Properties of Column Sections.

Properties of Two Channels Laced.



Flanges Turned Out.

Channels.		Total Area.	Moments of Inertia and Radii of Gyration.								Web of Channel.	Gages.		Max. Rivet.
			Axis A-A.		Axis B-B. Distance Inside to Inside of Webs in Inches = b′.									
Depth.	Weight.				4¼		5¼		6¼					
			I_A	r_A	I_B	r_B	I_B	r_B	I_B	r_B	t	d	h	
In.	Lb.	In.²	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.	In.	In.	In.
7	9.80	5.70	42	2.72	43	2.73	59	3.22	79	3.72	¼	1	1¼	⅝
"	12.25	7.16	48	2.59	51	2.65	71	3.14	95	3.64	5/16	"	1 5/16	"
					4½		5½		6½					
8	11.50	6.70	65	3.10	47	2.65	66	3.14	88	3.63	¼	1	1¼	¾
"	13.75	8.04	72	2.98	53	2.57	76	3.06	102	3.55	5/16	"	1 5/16	"
"	16.25	9.52	80	2.89	57	2.45	82	2.94	112	3.43	7/16	"	1 7/16	"
					6¼		7¼		8¼					
9	13.40	7.78	95	3.49	98	3.55	127	4.04	160	4.54	¼	1⅛	1⅜	¾
"	15.00	8.78	102	3.40	106	3.47	138	3.95	175	4.45	5/16	"	1 7/16	"
"	20.00	11.72	122	3.21	131	3.34	172	3.83	220	4.32	7/16	"	1 9/16	"
					6		7		8					
10	15.30	8.92	134	3.87	107	3.46	140	3.95	176	4.44	¼	1¼	1½	¾
"	20.00	11.72	157	3.66	129	3.31	170	3.80	217	4.29	⅜	"	1⅝	"
"	25.00	14.66	182	3.52	150	3.19	199	3.68	256	4.17	½	"	1¾	"
					8		9		10					
12	20.70	12.06	256	4.61	240	4.47	296	4.96	358	5.45	5/16	1½	1 13/16	⅞
"	25.00	14.64	288	4.43	281	4.37	348	4.87	423	5.36	⅜	"	1⅞	"
"	35.00	20.52	358	4.17	353	4.14	441	4.63	541	5.13	⅝	"	2⅛	"
					9½		10½		11½					
15	33.90	19.80	625	5.62	510	5.22	646	5.68	753	6.18	7/16	1¾	2 3/16	⅞
"	45.00	26.34	748	5.32	660	4.99	796	5.48	946	5.98	⅝	"	2⅜	"
"	55.00	32.22	858	5.16	758	4.84	920	5.33	1098	5.83	13/16	"	2 9/16	"

The table given above is intended to serve only as a guide in the choice of sections, and not as a complete table. The properties of sections not given in table may be found as follows:

Example.—Required the properties of a section consisting of 2 [s 10 in. at 15.3 lb., laced, with flanges turned out, 8¼ in. back to back. Distance inside to inside of web = 8¼ + ½ = 8¾″.

From Table 14, Area = 8.92 in.².

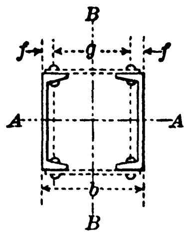
$I_A = I_X$ in Table 19 = 133.8 in.⁴; $r_A = \sqrt{I_A \div A} = \sqrt{133.8 \div 8.92} = 3.87$ in.

$I_B = I_Y$ in Table 19 = 207.0 in.⁴; $r_B = \sqrt{I_B \div A} = \sqrt{207.0 \div 8.92} = 4.81$ in.

TABLE 58.

Properties of Column Sections.

Properties of Two Channels Laced.



Flanges Turned In.

Channels.		Total Area.	Moments of Inertia and Radii of Gyration.											
			Axis A–A.		Axis B–B. Distance Back to Back of Channels in Inches = b.									
Depth.	Wt.				7½		8½		9½		10½		11½	
			I_A	r_A	I_B	r_B	I_B	r_B	I_B	r_B	I_B	r_B	I_B	r_B
In.	Lb.	In.²	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.⁴	In.
7	9.80	5.70	42.2	2.72	60.5	3.26	80.2	3.75	102.7	4.24	128.1	4.74	156.3	5.24
"	12 25	7.16	48.4	2 59	77.1	3.27	102.1	3.77	130.7	4 26	162.9	4.76	198.7	5.27
					7½		8½		9½		10½		11½	
8	11.50	6.70	64.6	3.10	70.2	3.24	93.1	3.73	119.4	4.22	149.0	4.72	182.0	5.21
"	13.75	8.04	72.0	2.98	85.5	3.25	113.3	3.74	145.2	4.23	181.1	4.73	221.0	5.23
					8½		9½		10½		11½		12½	
9	13.40	7.78	94.6	3.49	106.8	3.70	137.1	4.20	171.2	4.69	209.3	5.18	251.3	5.68
"	15.00	8.78	101.8	3.40	122.0	3.72	156.5	4.21	195.4	4 71	238.7	5.20	286.4	5.70
"	20.00	11.72	121.6	3.21	162.9	3.72	208.9	4.22	260.8	4.71	318.6	5.20	382 3	5.70
					9½		10½		11½		12½		13½	
10	15.30	8.92	133.8	3.87	1[illegible]5.3	4.17	194.2	4.68	237.6	5.16	285.4	5.66	337.7	6.15
"	20.00	11.72	157.4	3.66	207.4	4.20	259.0	4.69	316.5	5.19	379.9	5.68	449.2	6.18
"	25.00	14.66	182.0	3.52	257.5	4.18	321.9	4.68	393.7	5.18	472.8	5.67	559.2	6.17
					10½		11½		12½		13½		14½	
12	20.70	12.06	256.2	4.61	277.1	4.62	314.9	5.11	378.8	5.59	448.7	6.10	524.6	6.59
"	25.00	14.64	288.0	4.43	316.3	4.64	387.2	5.13	465.4	5.62	551.0	6.12	644.0	6.62
"	30.00	17.58	323.4	4.28	379.3	4.63	464.4	5.13	558.3	5.63	661.0	6.12	772.5	6.62
"	35.00	20.52	358.6	4.17	439.0	4.62	537.9	5.12	647.1	5.61	766.6	6.10	896.4	6.60
					12½		13½		14½		15½		16½	
15	33.90	19.80	625.2	5.62	605.9	5.53	718.9	6.02	841.7	6.52	974.5	7.02	1117.2	7.51
"	35.00	20.46	640.0	5.57	630.7	5.54	748.2	6.03	876.0	6.52	1014 2	7.02	1162.6	7.52
"	40.00	23.40	695.0	5.44	721.7	5.54	856.2	6.03	1002.4	6.51	1160.4	7.03	1330.2	7.52
"	45.00	26.34	747.8	5.32	810.6	5.53	961.9	6.02	1126.4	6.52	1304.1	7.02	1495.1	7.52

The table given above is intended to serve only as a guide in the choice of sections, and not as a complete table. The properties of sections not given in table may be found as follows:

Example 1: Required the properties of a section consisting of 2 [s 10 in. at 15.3 lb., laced, with flanges turned in, 10½ in. back to back.

From Table 14, Area = 8.92 in.².

$I_A = I_X$ from Table 20 = 133.8 in.⁴; $r_A = \sqrt{I_A \div A} = \sqrt{133.8 \div 8.92} = 3.87$ in.

$I_B = I_Y$ from Table 20 = 194 2 in.⁴; $r_B = \sqrt{I_B \div A} = \sqrt{194.2 \div 8.92} = 4.68$ in.

Example 2: Required the properties of a section consisting of 2 [s 10 in. at 15.3 lb., laced, with flanges turned in, 12 in. inside to inside of web.

From Table No. 14, Area = 8.92 in.².

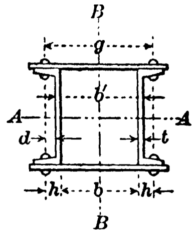
$I_A = I_X$ from Table 21 = 133.8 in.⁴; $r_A = \sqrt{I_A + A} = \sqrt{133.8 + 8.92} = 3.87$ in.

$I_B = I_Y$ from Table 21 = 284.4 in.⁴; $r_B = \sqrt{I_B + A} = \sqrt{284.4 + 8.92} = 5.65$ in.

TABLE 59.

PROPERTIES OF COLUMN SECTIONS.

Properties of Two Channels and Two Plates.



Flanges Turned Out.

Channels. Depth.	Channels. Weight.	Cover Plates.	Total Area.	Inside to Inside of Web.	Back to Back.	Moments of Inertia and Radii of Gyration. Axis A–A.		Axis B–B.		Gages. Plate.	Gages. Channels.	Web of Channel.	Max Rivet.
				b′	b	I_A	r_A	I_B	r_B	g	h	t	
In.	Lb.	In.	In.²	In.	In.	In.⁴	In.	In.⁴	In.	In.	In.	In.	In.
7	9.80	10×¼	10.70	5¾	5¼	108	3.18	101	3.07	7¾	1¼	¼	⅝
“	“	“ ⅜	13.20	“	“	144	3.31	122	3.04	“	“	“	“
“	12.25	“ ¼	12.16	“	5⅛	114	3.06	113	3.04	“	1 5/16	5/16	“
“	“	“ ⅜	14.66	“	“	150	3.20	134	3.02	“	“	“	“
8	11.50	12×¼	12.70	7½	7	167	3.62	186	3.83	9½	1¼	¼	¾
“	“	“ ⅜	15.70	“	“	223	3.76	222	3.76	“	“	“	“
“	13.75	“ ¼	14.04	“	6⅞	174	3.52	204	3.81	“	1 5/16	5/16	“
“	“	“ ⅜	17.04	“	“	230	3.67	240	3.74	“	“	“	“
9	13.40	12×⅜	16.78	7¼	6¾	293	4.17	235	3.74	9½	1⅜	¼	¾
“	“	“ ½	19.78	“	“	366	4.30	271	3.70	“	“	“	“
“	20.00	“ ⅜	20.72	“	6⅜	320	3.92	280	3.67	“	1 9/16	7/16	“
“	“	“ ½	23.72	“	“	393	4.06	316	3.64	“	“	“	“
10	15.30	14×⅜	19.42	9	8½	417	4.63	389	4.47	11½	1½	¼	¾
“	“	“ ⅝	26.42	“	“	628	4.88	504	4.37	“	“	“	“
“	25.00	“ ⅜	25.16	“	8	465	4.29	492	4.42	“	1¾	½	“
“	“	“ ⅝	32.16	“	“	676	4.58	606	4.34	“	“	“	“
12	20.70	16×⅜	24.06	10	9⅜	715	5.45	614	5.05	13	1 13/16	5/16	⅞
“	“	“ ⅝	32.06	“	“	1053	5.73	785	4.95	“	“	“	“
“	25.00	“ ⅜	26.64	“	9¼	747	5.29	679	5.04	“	1⅞	⅜	“
“	“	“ ⅝	34.64	“	“	1085	5.59	849	4.94	“	“	“	“
“	35.00	“ ½	36.52	“	8¾	984	5.19	882	4.91	“	2⅛	⅝	“
“	“	“ ¾	44.52	“	“	1335	5.47	1053	4.86	“	“	“	“
15	33.90	18×⅜	33.30	11½	10⅝	1423	6.54	1119	5.79	15	2 3/16	7/16	⅞
“	“	“ ⅝	42.30	“	“	1999	6.87	1362	5.68	“	“	“	“
“	45.00	“ ⅜	39.84	“	10¼	1548	6.22	1311	5.72	“	2⅜	⅝	“
“	“	“ ⅝	48.84	“	“	2124	6.59	1554	5.63	“	“	“	“
“	55.00	“ ½	50.22	“	9⅞	1942	6.21	1584	5.61	“	2 9/16	13/16	“
“	“	“ ¾	59.22	“	“	2536	6.54	1827	5.55	“	“	“	“

The table given above is intended to serve only as a guide in the choice of sections, and not as a complete table. The properties of sections not given in table may be found as follows:

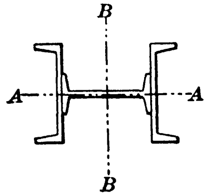
Example: Required the properties of a section consisting of 2 [s 12 in. at 20.7 lb., flanges turned out, 9¼ in. back to back, and 2 Pls. 16″×½″.

Item. Number.	Item. Section.	Item. Size.	A Table.	A In.²	I_A Table.	I_A In.⁴	I_B Table.	I_B In.⁴	r_A In.	r_B In.
2	[s	12 in. at 20.7	14	12.06	19	256	19	350	$\sqrt{\frac{882}{28.06}}$	$\sqrt{\frac{691}{28.06}}$
2	Pls	16″×½″	1	16.00	5	626	3	341		
		Total		28.06		882		691	5.61	4.96

TABLE 60.

PROPERTIES OF COLUMN SECTIONS.

Properties of Channel and I-Beam Section.



Channel Flanges Out. Minimum I-Beam for Web.

SERIES I AND II.		SERIES I.							SERIES II.						
Flange Channels.		Web Beam.		Total Area.	Moments of Inertia and Radii of Gyration.				Web Beam.		Total Area.	Moments of Inertia and Radii of Gyration.			
					Axis A–A.		Axis B–B.					Axis A–A.		Axis B–B.	
Depth.	Weight.	Depth.	Weight.		I_A	r_A	I_B	r_B	Depth.	Weight.		I_A	r_A	I_B	r_B
In.	Lb.	In.	Lb.	In.²	In.⁴	In.	In.⁴	In.	In.	Lb.	In.²	In.⁴	In.	In.⁴	In.
6	8.20	6	12.50	8.37	28	1.82	82	3.13	7	15.30	9.18	29	1.77	114	3.53
"	10.50	"	"	9.75	32	1.81	99	3.19	"	"	10.57	33	1.76	137	3.59
7	9.80	6	12.50	9.31	44	2.18	95	3.20	7	15.30	10.12	45	2.11	131	3.60
"	12.25	"	"	10.77	50	2.16	114	3.24	"	"	11.59	51	2.10	155	3.66
8	11.50	6	12.50	10.31	66	2.54	110	3.27	7	15.30	11.12	67	2.46	150	3.67
"	13.75	"	"	11.65	74	2.51	127	3.30	"	"	12.47	75	2.44	172	3.71
9	13.40	7	15.30	12.20	97	2.82	171	3.74	8	18.40	13.11	98	2.74	226	4.15
"	15.00	"	"	13.21	104	2.81	188	3.76	"	"	14.12	106	2.73	247	4.17
"	20.00	"	"	16.15	124	2.77	237	3.83	"	"	17.06	125	2.71	309	4.25
10	15.30	8	18.40	14.28	138	3.11	253	4.22	9	21.80	15.23	139	3.02	325	4.62
"	20.00	"	"	17.06	161	3.07	312	4.28	"	"	18.04	163	3.00	398	4.69
"	25.00	"	"	20.00	186	3.05	377	4.34	"	"	20.98	187	2.98	477	4.77
12	20.7	9	21.80	18.38	261	3.77	419	4.78	10	25.40	19.43	263	3.68	522	5.18
"	25.00	"	"	20.96	293	3.74	488	4.82	"	"	22.02	295	3.66	605	5.24
"	30.00	"	"	23.90	329	3.70	568	4.87	"	"	24.96	330	3.63	701	5.29
"	35.00	"	"	26.84	364	3.68	652	4.92	"	"	27.90	366	3.62	801	5.35
"	40.00	"	"	29.78	399	3.66	740	4.98	"	"	30.84	401	3.60	905	5.41
15	33.90	10	25.40	27.18	632	4.82	803	5.44	12	31.80	29.06	635	4.67	1146	6.28
"	35.00	"	"	27.84	647	4.81	829	5.45	"	"	29.72	650	4.67	1181	6.29
"	40.00	"	"	30.78	702	4.77	927	5.48	"	"	32.66	705	4.64	1317	6.34
"	45.00	"	"	33.72	757	4.73	1030	5.52	"	"	35.60	760	4.61	1457	6.38
"	50.00	"	"	36.66	812	4.70	1135	5.55	"	"	38.54	815	4.59	1600	6.43
"	55.00	"	"	39.60	867	4.67	1244	5.60	"	"	41.48	870	4.57	1747	6.48

The table given above is intended to serve only as a guide in the choice of sections, and not as a complete table. The properties of sections not given in the table may be found as follows:

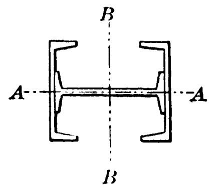
Example: Required the properties of a section consisting of 2 [s 10 in. at 20.7 lb., flanges turned out, and one I 9 in. at 21.8 lb.

Item.			A		I_A		I_B		r_A	r_B
Number.	Section.	Size.	Table.	In.²	Table.	In.⁴	Table.	In.⁴	In.	In.
2	[s	10 in. at 20.7 lb.	14	11.76	19	157.4	19	312.7	$\sqrt{\frac{162.6}{18.07}}$	$\sqrt{\frac{397.6}{18.07}}$
1	I	9 in. at 21.8 lb.	7	6.31	7	5.2	7	84.9		
		Total		18.07		162.6		397.6	3.00	4.69

TABLE 61.

Properties of Column Sections.

Properties of Channel and I-Beam Section.



Channel Flanges In. Minimum I-Beam for Web.

Series I and II.		Series I.							Series II.						
Flange Channels.		Web Beam.		Total Area.	Moments of Inertia and Radii of Gyration.				Web Beam.		Total Area.	Moments of Inertia and Radii of Gyration.			
					Axis A-A.		Axis B-B.					Axis A-A.		Axis B-B.	
Depth.	Weight.	Depth.	Weight.		I_A	r_A	I_B	r_B	Depth.	Weight.		I_A	r_A	I_B	r_B
In.	Lb.	In.	Lb.	In.²	In.⁴	In.	In.⁴	In.	In	Lb	In.²	In.⁴	In.	In.⁴	In.
6	8.20	7	15.30	9.21	29	1.77	86	3.06	8	18.40	10.12	30	1.72	123	3.49
"	10.50	"	"	10.57	33	1.76	106	3.16	"	"	11.48	34	1.72	149	3.60
7	9.80	7	15.30	10.12	45	2.11	95	3.07	8	18.40	11.04	46	2.04	135	3.50
"	12.25	"	"	11.59	51	2.10	117	3.17	"	"	12.50	52	2.04	163	3.61
8	11.50	8	18.40	12.06	68	2.38	149	3.52	9	21.80	13.04	70	2.32	203	3.95
"	13.75	"	"	13.38	76	2.38	174	3.60	"	"	14.36	77	2.32	234	4.03
9	13.40	9	21.80	14.10	100	2.66	221	3.96	10	25.40	15.16	101	2.58	292	4.39
"	15.00	"	"	15.10	107	2.66	244	4.02	"	"	16.16	109	2.59	321	4.45
"	20.00	"	"	18.04	127	2.65	314	4.17	"	"	19.10	129	2.60	405	4.60
10	15.30	9	21.80	15.26	139	3.02	240	3.97	10	25.40	16.32	141	2.94	316	4.40
"	20.00	"	"	18.04	163	3.00	305	4.11	"	"	19.10	164	2.93	396	4.55
"	25.00	"	"	20.98	187	2.98	378	4.24	"	"	22.04	189	2.93	483	4.68
12	20.70	10	25.40	19.44	263	3.68	383	4.44	12	31.80	21.32	266	3.53	599	5.30
"	25.00	"	"	22.02	295	3.66	458	4.55	"	"	23.90	298	3.52	705	5.42
"	30.00	"	"	24.96	330	3.63	545	4.67	"	"	26.84	333	3.52	827	5.54
"	35.00	"	"	27.90	366	3.62	637	4.77	"	"	29.78	368	3.51	954	5.66
"	40.00	"	"	30.84	401	3.60	732	4.87	"	"	32.72	404	3.51	1086	5.76
15	33.90	12	31.80	29.06	635	4.67	855	5.42	15	42.90	32.29	640	4.45	1458	6.72
"	35.00	"	"	29.72	650	4.67	887	5.45	"	"	32.95	655	4.45	1507	6.75
"	40.00	"	"	32.66	705	4.64	1010	5.55	"	"	35.89	710	4.44	1694	6.86
"	45.00	"	"	35.60	760	4.61	1138	5.64	"	"	38.83	765	4.43	1887	6.96
"	50.00	"	"	38.54	815	4.59	1268	5.73	"	"	41.77	820	4.42	2083	7.05
"	55.00	"	"	41.48	870	4.57	1403	5.81	"	"	44.71	875	4.41	2284	7.15

The table given above is intended to serve only as a guide in the choice of sections, and not as a complete table. The properties of sections not given in the table may be found as follows:

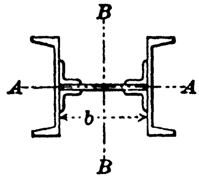
Example: Required the properties of a section consisting of 2 [s 10 in. at 20 lb., flanges turned in and one I9 in. at 21.9 lb.

Item.			A		I_A		I_B		r_A	r_B
Number.	Section.	Size.	Table.	In.²	Table.	In.⁴	Table.	In.⁴	In.	In.
2	[s	10 in. at 20 lb.	14	11.72	21	157.0	21	220.2	$\sqrt{\frac{162.2}{18.04}}$	$\sqrt{\frac{305.1}{18.04}}$
1	I	9 in. at 21.8 lb.	7	6.32	7	5.2	7	84.9		
		Total		18.04		162.2		305.1	3.00	4.11

TABLE 62.

PROPERTIES OF TWO CHANNELS AND A BUILT I-BEAM. FLANGES TURNED OUT.

Properties of Two Channels and a Built I-Beam.



Channel Flanges Out, Distance Back to Back of Channels Equals Width of Web Plate Plus ¼″.

Series 1 and 2.			Series 1.						Series 2.					
Channel.		Size of Angles.	Size of Web Plate.	Total Area.	Axis A-A.		Axis B-B.		Size of Web Plate.	Total Area.	Axis A-A.		Axis B-B.	
Depth.	Weight.				Moment of Inertia.	Radius of Gyration.	Moment of Inertia.	Radius of Gyration.			Moment of Inertia.	Radius of Gyration.	Moment of Inertia.	Radius of Gyration.
				A	I_A	r_A	I_B	r_B		A	I_A	r_A	I_B	r_B
In.	Lb.	In.	In.	In.^{2}	In.^{4}	In.	In.^{4}	In.	In.	In.^{2}	In.^{4}	In.	In.^{4}	In.
12	20.7	3x3x$\frac{5}{16}$	8x$\frac{1}{4}$	21.18	269	3.57	402	4.35	10x$\frac{3}{8}$	22.93	270	3.44	610	5.16
12	25	"	"	23.76	301	3.56	464	4.41	"	25.51	302	3.44	700	5.23
12	30	"	"	26.70	337	3.55	536	4.48	"	28.45	337	3.44	804	5.31
12	20.7	3½x3½x$\frac{3}{8}$	8x$\frac{3}{8}$	24.98	282	3.36	436	4.18	10x$\frac{3}{8}$	25.73	282	3.31	657	5.05
12	25	"	"	27.56	314	3.37	498	4.25	"	28.31	314	3.32	747	5.13
12	30	"	"	30.50	349	3.37	571	4.33	"	31.25	349	3.33	851	5.21
15	33.90	3½x3½x$\frac{3}{8}$	8x$\frac{3}{8}$	32.72	651	4.46	652	4.46	10x$\frac{3}{8}$	33.47	651	4.41	961	5.36
15	35	"	"	33.38	666	4.46	672	4.48	"	34.13	666	4.41	989	5.38
15	40	"	"	36.32	721	4.45	747	4.53	"	37.07	721	4.41	1096	5.43
15	33.90	4x4x$\frac{3}{8}$	10x$\frac{3}{8}$	34.99	663	4.35	982	5.30	12x$\frac{3}{8}$	35.74	663	4.31	1110	5.57
15	35	"	"	35.65	677	4.35	1010	5.32	"	36.40	677	4.31	1138	5.58
15	40	"	"	38.59	733	4.35	1117	5.37	"	39.34	733	4.31	1245	5.62

The above table is intended to serve only as a guide in the choice of sections and not as a complete table. The properties of sections not given in table may be obtained as follows:

Example: Determine the properties of a section composed of 2 channels 15″ × 55 lb., 1 plate 12″ × ½″ and 4 angles 4″ × 4″ × ½″, 12¼″ back to back.

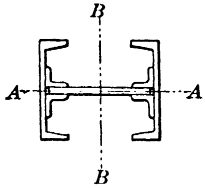
Solution:

Item.	Area.		Moment of Inertia.				Radius of Gyration.	
			Axis A-A.		Axis B-B.		Axis A-A.	Axis B-B.
	Table No.	A	Table No.	I_A	Table No.	I_B	$r_A = \sqrt{I_A \div A}$	$r_B = \sqrt{I_B \div A}$
		In.^{2}		In.^{4}		In.^{4}	In.	In.
2 [s 15″x55 lb.	14	32.22	19	858	19	1587	$\sqrt{\frac{911}{53.22}}$	$\sqrt{\frac{2048}{53.22}}$
1 Pl—12″x½″	1	6.00	4	0	3	72		
4∠4″x4″x½″	32	15.00	35	53	32	389		
Total.......	A =	53.22	I_A =	911	I_B =	2048	r_A = 4.14	r_B = 6.20

TABLE 63.

Properties of Two Channels and a Built I-Beam.
Flanges Turned In.

Properties of Two Channels and a Built I-Beam.



Channel Flanges In. Distance Inside to Inside Of Channels Equals Width of Web Plate Plus ¼″.

Series 1 and 2. Channels. Depth.	Weight.	Size of Angles.	Series 1. Size of Web Plate.	Total Area. A	Axis A-A. Moment of Inertia. I_A	Radius of Gyration. r_A	Axis B-B. Moment of Inertia. I_B	Radius of Gyration. r_B	Series 2. Size of Web Plate.	Total Area. A	Axis A-A. Moment of Inertia. I_A	Radius of Gyration. r_A	Axis B-B. Moment of Inertia. I_B	Radius of Gyration. r_B
In.	Lb.	In.	In.	In.^{2}	In.^{4}	In.	In.^{4}	In.	In.	In.^{2}	In.^{4}	In.	In.^{4}	In.
12	20.7	3x3x$\frac{5}{16}$	10x¼	21.68	269	3.52	4 3	4.57	12x⅜	23.68	270	3.38	683	5.38
12	25	"	"	24.26	301	3.52	535	4.70	"	26.26	302	3.38	798	5.53
12	30	"	"	27.20	336	3.52	631	4.81	"	29.20	337	3.39	930	5.64
12	20.7	3½x3½x⅜	14x⅜	27.23	282	3.22	1054	6.22	16x½	29.98	283	3.08	1449	6.93
12	25	"	"	29.81	314	3.24	1205	6.35	"	32.56	315	3.11	1644	7.10
12	30	"	"	32.75	349	3.25	1380	6.49	"	35.50	350	3.14	1867	7.25
15	33.9	3½x3½x⅜	12x⅜	34.22	651	4.36	1034	5.50	14x⅜	34.97	651	4.31	1431	6.40
15	35	"	"	34.88	666	4.36	1068	5.52	"	35.63	666	4.32	1477	6.43
15	40	"	"	37.82	721	4.36	1201	5.63	"	38.57	721	4.32	1652	6.54
15	33.9	4x4x⅜	16x⅜	37.24	663	4.22	1963	7.26	18x½	40.24	667	4.07	2582	8.01
15	35	"	"	37.90	677	4.22	2021	7.29	"	40.90	679	4.07	2655	8.05
15	40	"	"	40.84	733	4.23	2245	7.41	"	43.84	735	4.09	2933	8.18

The above table is intended to serve only as a guide in the choice of sections and not as a complete table. The properties of sections not given in table may be obtained as follows:

Example: Determine the properties of a section composed of 2 channels 15″ × 55 lb., 1 plate 18″ × ⅝″ and 4 angles 4″ × 4″ × ½″, 18¼″ back to back.

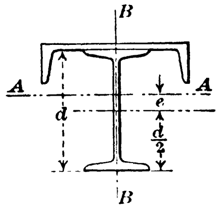
Solution:

Item.	Area. Table No.	A (In.^{2})	Moment of Inertia. Axis A-A. Table No.	I_A (In.^{4})	Axis B-B. Table No.	I_B (In.^{4})	Radius of Gyration. Axis A-A. $r_A = \sqrt{I_A \div A}$ (In.)	Axis B-B. $r_B = \sqrt{I_B \div A}$ (In.)
2[s 15″x55 lb.	21	32.22	21	858	21	2716	$\sqrt{\frac{914}{58.47}}$	$\sqrt{\frac{3989}{58.47}}$
1 Pl—18″x⅝″	1	11.25	4	0	3	304		
4 ∠ 4″x4″x½″	32	15.00	35	56	32	969		
Total.....	A =	58.47	I_A =	914	I_B =	3989	r_A = 3.96	r_B = 8.25

TABLE 64.

PROPERTIES OF ONE CHANNEL AND ONE I-BEAM.

Properties of One Channel and One I-Beam.



Properties of One Channel and One I-Beam.

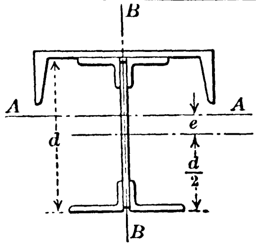
Ser. 1 & 2. Beam. Depth.	Ser. 1 & 2. Beam. Weight.	Series 1. Channel. Depth.	Series 1. Channel. Weight.	Series 1. Total Area. A	Series 1. Axis A–A. Moment of Inertia. I_A	Series 1. Axis A–A. Radius of Gyration. r_A	Series 1. Axis A–A. Eccentricity. e	Series 1. Axis B–B. Moment of Inertia. I_B	Series 1. Axis B–B. Radius of Gyration. r_B	Series 2. Channel. Depth.	Series 2. Channel. Weight.	Series 2. Total Area. A	Series 2. Axis A–A. Moment of Inertia. I_A	Series 2. Axis A–A. Radius of Gyration. r_A	Series 2. Axis A–A. Eccentricity. e	Series 2. Axis B–B. Moment of Inertia. I_B	Series 2. Axis B–B. Radius of Gyration. r_B
In.	Lb.	In.	Lb.	$In.^2$	$In.^4$	In.	In.	$In.^4$	In.	In.	Lb.	$In.^2$	$In.^4$	In.	In.	$In.^4$	In.
8	18	5	6½	7.28	77	3.25	0.99	11.2	1.24	6	8	7.71	80	3.22	1.13	16.8	1.48
"	20½	"	"	7.91	81	3.20	0.91	11.4	1.20	"	"	8.41	84	3.16	1.04	17.0	1.42
9	21	6	8	8.69	116	3.65	1.15	18.2	1.45	8	11¼	9.66	124	3.58	1.44	37.5	1.97
"	25	"	"	9.73	124	3.57	1.02	18.6	1.38	"	"	10.70	133	3.52	1.30	37.9	1.88
10	25	6	8	9.75	162	4.08	1.14	19.9	1.43	8	11¼	10.72	173	4.02	1.45	39.2	1.91
"	30	"	"	11.20	176	3.97	0.99	20.6	1.36	"	"	12.17	188	3.92	1.28	39.9	1.81
12	31½	8	11¼	12.61	295	4.84	1.50	41.8	1.82	10	15	13.72	313	4.77	1.82	76.4	2.36
"	40	"	"	15.19	353	4.82	1.25	46.1	1.74	"	"	16.30	373	4.78	1.53	80.7	2.22
15	42	8	11¼	15.83	578	6.04	1.51	46.9	1.72	10	15	16.94	610	6.00	1.87	81.5	2.19
"	"	12	20½	18.51	649	5.92	2.31	142.7	2.78	15	33	22.38	729	5.71	3.15	327.2	3.82
"	50	8	11¼	18.06	624	5.88	1.32	48.3	1.63	10	15	19.17	658	5.86	1.65	82.9	2.08
"	"	12	20½	20.74	702	5.81	2.06	144.1	2.64	15	33	24.61	791	5.67	2.86	328.6	3.65
"	60	8	11¼	21.00	754	5.97	1.14	58.3	1.67	10	15	22.13	791	5.98	1.43	92.9	2.05
"	"	12	20½	23.68	838	5.95	1.80	154.1	2.55	15	33	27.57	938	5.83	2.55	338.6	3.50
18	55	8	11¼	19.28	1004	7.21	1.50	53.5	1.67	10	15	20.39	1056	7.19	1.88	88.1	2.08
"	"	12	20½	21.96	1122	7.14	2.35	149.3	2.61	15	33	25.83	1257	6.97	3.30	333.8	3.59
"	65	8	11¼	22.47	1096	6.98	1.28	55.8	1.58	10	15	23.58	1151	6.98	1.63	90.4	1.96
"	"	12	20½	25.15	1223	6.97	2.06	151.6	2.46	15	33	29.02	1373	6.88	2.94	336.1	3.40
"	75	8	11¼	25.40	1360	7.32	1.14	78.7	1.76	10	15	26.51	1418	7.31	1.45	113.1	2.06
"	"	12	20½	28.08	1494	7.29	1.84	174.3	2.49	15	33	31.95	1656	7.24	2.67	358.8	3.37
20	65	9	13¼	22.97	1470	8.00	1.63	75.2	1.81	10	15	23.54	1507	8.00	1.82	94.8	2.01
"	"	12	20½	25.11	1594	7.97	2.30	156.0	2.49	15	33	28.98	1779	7.84	3.29	340.5	3.43
"	70	9	13¼	24.48	1524	7.89	1.53	76.3	1.77	10	15	25.05	1562	7.89	1.71	95.9	1.96
"	"	12	20½	26.62	1652	7.88	2.17	157.1	2.43	15	33	30.49	1846	7.79	3.12	341.6	3.34
"	80	9	13¼	27.62	1777	8.02	1.36	93.1	1.84	10	15	28.19	1816	8.03	1.52	112.7	2.00
"	"	12	20½	29.76	1912	8.02	1.94	173.9	2.42	15	33	33.63	2120	7.94	2.83	358.4	3.26
24	80	9	13¼	27.21	2539	9.66	1.66	90.2	1.82	10	15	27.78	2594	9.66	1.86	109.8	1.99
"	"	12	20½	29.35	2734	9.66	2.38	171.0	2.41	15	33	33.22	3033	9.55	3.46	355.5	3.27
"	90	9	13¼	30.36	2700	9.43	1.49	93.0	1.75	10	15	30.93	2755	9.43	1.67	112.6	1.91
"	"	12	20½	32.50	2902	9.45	2.15	173.8	2.31	15	33	36.37	3219	9.40	3.16	358.3	3.14
"	100	10	15	33.87	2904	9.26	1.53	115.5	1.85	15	40	41.17	3548	9.28	3.35	396.1	3.10
"	"	12	20½	35.44	3055	9.29	1.97	176.7	2.23	15	33	39.31	3387	9.28	2.92	361.2	3.03
"	105	10	15	35.44	3338	9.69	1.46	145.8	2.03	15	40	42.74	3997	9.67	3.23	426.4	3.16
"	"	12	20½	37.01	3492	9.71	1.89	207.0	2.36	15	33	40.88	3831	9.67	2.81	391.5	3.09

Note 1924. This table was calculated in 1914. Minimum channels have been used for which all values are as given in Table 14 except "weights." For minimum I-beams given in Table 7 all properties given in this table, except "weights" are exact. For other than minimum I-beams check "total areas"; other values are correct within two-tenths of one per cent.

TABLE 65.

Properties of One Channel and a Built I-Beam.

Properties of One Channel and One Built I-Beam.



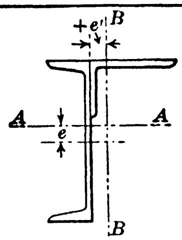
Back to Back of Angles Equals Width of Web Plate Plus ½″
*** Top Angles, Short Legs Out. Bottom Angles, Long Legs Out.**

Plate.	Channel.		Angles.		Total Area.	Axis A–A.			Axis B–B.	
Web.	Depth.	Weight.	Bottom.	Top.		Moment of Inertia.	Radius of Gyration.	Eccentricity.	Moment of Inertia.	Radius of Gyration.
					A	I_A	r_A	e	I_B	r_B
In.	In.	Lb.	In.	In.	$In.^2$	$In.^4$	In.	In.	$In.^4$	In.
16x⅜	10	15.3	5x3½x⅜	3½x3½x⅜	21.52	979	6.75	1.20	115	2.31
"	"	"	" ½	" ½	24.96	1166	6.83	0.92	132	2.30
"	"	"	" ⅝	" ⅝	28.26	1340	6.89	0.71	148	2.29
"	12	20.7	6x4x⅜	4x4x⅜	24.97	1144	6.77	1.41	207	2.87
"	"	"	" ½	" ½	29.03	1367	6.86	1.08	233	2.84
"	"	"	" ⅝	" ⅝	32.97	1572	6.91	0.83	260	2.81
18x½	10	15.3	5x3½x⅜	3½x3½x⅜	24.52	1338	7.39	1.19	117	2.19
"	"	"	" ½	" ½	27.96	1577	7.51	0.92	134	2.19
"	"	"	" ⅝	" ⅝	31 26	1802	7.59	0.72	152	2.20
"	12	20.7	6x4x⅜	4x4x⅜	27.97	1555	7.46	1.42	209	2.73
"	"	"	" ½	" ½	32.03	1838	7.58	1.10	237	2.72
"	"	"	" ⅝	" ⅝	35.97	2103	7.64	0.86	265	2.71
20x½	12	20.7	6x4x⅜	4x4x⅜	28.97	1971	8.24	1.52	209	2.69
"	"	"	" ½	" ½	33.03	2329	8.39	1.19	237	2.68
"	"	"	" ⅝	" ⅝	36.97	2662	8.49	0.93	265	2.68
"	15	33.9	6x6x⅜	*6x4x⅜	35.84	2317	8.04	2.30	395	3.32
"	"	"	" ½	" ½	40.90	2725	8.16	1.90	423	3.22
"	"	"	" ⅝	" ⅝	45.84	3104	8.24	1.59	451	3.14
24x⅝	12	20.7	6x4x⅜	4x4x⅜	33.97	3133	9.62	1.56	212	2.50
"	"	"	" ½	" ½	38.03	3656	9.81	1.24	241	2.52
"	"	"	" ⅝	" ⅝	41.97	4150	9.95	0.99	270	2.54
"	15	33.9	6x6x⅜	*6x4x⅜	40.84	3686	9.50	2.12	398	3.12
"	"	"	" ½	" ½	45.90	4290	9.67	2.03	427	3.05
"	"	"	" ⅝	" ⅝	50.84	4858	9.78	1.72	457	3.00
30x¾	12	20.7	6x4x⅜	4x4x⅜	41.47	5546	11.56	1.61	217	2.29
"	"	"	" ½	" ½	45.53	6381	11.84	1.30	246	2.32
"	"	"	" ⅝	" ⅝	49.47	7174	12.05	1.05	276	2.36
"	15	33.9	6x6x½	*6x4x½	53.40	7490	11.84	2.19	432	2.85
"	"	"	" ⅝	" ⅝	58.34	8413	12.01	1.88	463	2.82
"	"	"	" ¾	" ¾	63.16	9293	12.13	1.63	495	2.80
36x¾	12	20.7	6x6x½	*6x4x½	54.03	10485	13.93	1.32	248	2.14
"	"	"	" ⅝	" ⅝	58.97	11825	14.16	1.06	278	2.17
"	"	"	" ¾	" ¾	63.79	13104	14.31	0.85	311	2.20
"	15	33.9	6x6x½	*6x4x½	57.90	11483	14.08	2.43	433	2.74
"	"	"	" ⅝	" ⅝	62.84	12859	14.31	2.10	463	2.72
"	"	"	" ¾	" ¾	67.66	14170	14.47	1.82	495	2.70

TABLE 66.

Properties of Built Struts.

Properties of One Channel and One Angle.



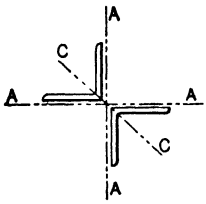
Long Leg of Angle Turned Out. Back of Angle Flush with Flange of Channel.

Depth of Channel.	Weight of Channel.	Size of Angle.	Total Area.	Axis A-A. Moment of Inertia.	Axis A-A. Radius of Gyration.	Axis A-A. Section Modulus.	Axis A-A. Eccentricity.	Axis B-B. Moment of Inertia.	Axis B-B. Radius of Gyration.	Axis B-B. Section Modulus.	Axis B-B. Eccentricity.
			A	I_A	r_A	S_A	e	I_B	r_B	S_B	e'
In.	Lb.	In.	In.^{2}	In.^{4}	In.	In.^{3}	In.	In.^{4}	In.	In.^{3}	In.
4	5.4	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}$	2.74	5.7	1.44	2.23	.56	1.97	.85	0.81	+.05
		$3\times2\frac{1}{2}\times\frac{1}{4}$	2.86	5.8	1.43	2.22	.62	2.82	.99	1.00	+.17
5	6.7	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}$	3.14	10.3	1.81	3.24	.68	2.27	.85	.90	−.03
		$3\times2\frac{1}{2}\times\frac{1}{4}$	3.26	10.8	1.82	3.34	.74	3.19	.99	1.09	+.07
		$3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}$	3.39	11.1	1.81	3.36	.80	4.41	1.14	1.33	+.19
		$4\times3\times\frac{5}{16}$	4.04	12.1	1.73	3.56	.90	6.96	1.31	1.94	+.41
6	8.2	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}$	3.57	17.8	2.23	4.74	.76	2.62	.86	1.01	−.11
		$3\times2\frac{1}{2}\times\frac{1}{4}$	3.69	18.3	2.23	4.78	.83	3.59	.99	1.19	−.01
		$3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}$	3.82	18.9	2.23	4.85	.90	4.89	1.13	1.44	+.09
		$4\times3\times\frac{5}{16}$	4.47	20.2	2.13	4.99	1.05	7.61	1.30	2.06	+.31
7	9.8	$3\times2\frac{1}{2}\times\frac{1}{4}$	4.16	29.1	2.64	6.62	.89	4.06	.99	1.31	−.09
		$3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}$	4.29	30.0	2.64	6.71	.97	5.42	1.12	1.55	+.01
		$4\times3\times\frac{5}{16}$	4.94	31.8	2.54	6.83	1.16	8.31	1.30	2.20	+.22
		$5\times3\times\frac{5}{16}$	5.25	33.2	2.51	6.94	1.29	13.73	1.62	3.03	+.47
8	11.5	$4\times3\times\frac{5}{16}$	5.44	47.5	2.95	9.06	1.24	9.07	1.29	2.34	+.13
		$5\times3\times\frac{5}{16}$	5.75	49.6	2.93	9.21	1.39	14.74	1.60	3.18	+.36
		$5\times3\frac{1}{2}\times\frac{5}{16}$	5.91	49.5	2.89	9.22	1.37	14.76	1.58	3.18	+.36
		$6\times3\frac{1}{2}\times\frac{3}{8}$	6.77	53.3	2.81	9.48	1.62	25.82	1.95	4.91	+.74
		$6\times4\times\frac{3}{8}$	6.96	53.4	2.77	9.56	1.59	25.87	1.93	4.91	+.73
9	13.4	$4\times3\times\frac{5}{16}$	5.98	68.0	3.37	11.70	1.31	9.91	1.29	2.50	+.04
		$5\times3\times\frac{5}{16}$	6.29	70.7	3.35	11.86	1.46	15.82	1.59	3.34	+.26
		$5\times3\frac{1}{2}\times\frac{5}{16}$	6.45	70.7	3.31	11.88	1.45	15.84	1.57	3.34	+.26
		$6\times3\frac{1}{2}\times\frac{3}{8}$	7.31	76.0	3.22	12.20	1.74	27.42	1.94	5.11	+.63
		$6\times4\times\frac{3}{8}$	7.50	76.0	3.18	12.23	1.71	27.46	1.91	5.10	+.62
10	15.3	$4\times3\times\frac{5}{16}$	6.55	94.1	3.79	14.81	1.35	10.82	1.28	2.68	−.03
		$5\times3\times\frac{5}{16}$	6.86	97.7	3.77	15.00	1.51	16.97	1.57	3.51	+.17
		$5\times3\frac{1}{2}\times\frac{5}{16}$	7.02	97.7	3.73	15.00	1.52	16.99	1.55	3.52	+.17
		$6\times3\frac{1}{2}\times\frac{3}{8}$	7.88	104.8	3.65	15.36	1.83	29.05	1.92	5.31	+.52
		$6\times4\times\frac{3}{8}$	8.07	104.6	3.60	15.35	1.82	29.10	1.90	5.31	+.52
12	20.7	$4\times3\times\frac{5}{16}$	8.12	172.3	4.61	23.45	1.35	13.25	1.28	3.16	−.20
		$5\times3\times\frac{5}{16}$	8.43	177.9	4.59	23.68	1.52	19.90	1.54	3.97	−.02
		$5\times3\frac{1}{2}\times\frac{5}{16}$	8.59	178.8	4.56	23.73	1.54	19.93	1.52	3.97	−.02
		$6\times3\frac{1}{2}\times\frac{3}{8}$	9.45	190.7	4.49	24.19	1.89	33.16	1.87	5.81	+.29
		$6\times4\times\frac{3}{8}$	9.64	190.8	4.45	24.19	1.90	33.15	1.85	5.81	+.29
15	33.9	$4\times3\times\frac{5}{16}$	11.99	392.6	5.72	45.25	1.18	18.86	1.25	4.26	−.43
		$5\times3\times\frac{5}{16}$	12.30	404.0	5.72	45.75	1.33	26.82	1.48	5.13	−.23
		$5\times3\frac{1}{2}\times\frac{5}{16}$	12.46	405.4	5.70	45.71	1.37	26.87	1.47	5.15	−.22
		$6\times3\frac{1}{2}\times\frac{3}{8}$	13.32	430.9	5.69	46.70	1.72	41.47	1.76	6.84	−.06
		$6\times4\times\frac{3}{8}$	13.51	431.3	5.66	46.65	1.75	41.47	1.75	6.84	−.06

TABLE 67.

Properties of Starred Angles.

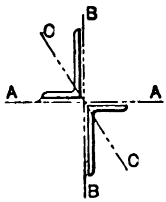
Two Angles Starred, Equal Legs.



Values for Axis A–A same as in Table 38.

Size of Angles.	Total Area.	Least Radius of Gyration.
	A	r_C
In.	In.²	In.
2x2x¼	1.88	.77
" ⅜	2.72	.74
2½x2½x¼	2.38	.97
" ⅜	3.46	.95
3x3x¼	2.88	1.17
" ⅜	4.22	1.16
" ½	5.50	1.13
" ⅝	6.72	1.10
3½x3½x¼	3.38	1.37
" ⅜	4.96	1.35
" ½	6.50	1.33
" ⅝	7.96	1.31
4x4x¼	3.88	1.58
" ⅜	5.72	1.56
" ½	7.50	1.53
" ⅝	9.22	1.51
5x5x⅜	7.22	1.98
" ½	9.50	1.95
" ⅝	11.72	1.92
" ¾	13.88	1.89
6x6x⅜	8.72	2.37
" ½	11.50	2.35
" ⅝	14.22	2.33
" ¾	16.88	2.30
" ⅞	19.46	2.28
" 1	22.00	2.26
8x8x½	15.50	3.17
" ⅝	19.22	3.14
" ¾	22.88	3.12
" ⅞	26.46	3.09
" 1	30.00	3.07

Two Angles Starred, Unequal Legs.

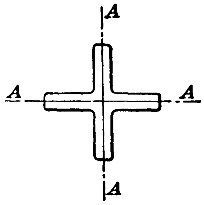


Values for Axes A–A & B–B same as in Tables 39 & 40 respectively.

Size of Angles.	Total Area.	Least Radius of Gyration.
	A	r_C
In.	In.²	In.
2½x2x¼	2.12	.73
" ⅜	3.10	.78
3x2½x¼	2.62	1.00
" ⅜	3.84	1.00
3½x3x¼	3.12	1.22
" ⅜	4.60	1.20
" ½	6.00	1.18
" ⅝	7.34	1.16
4x3x¼	3.38	1.23
" ⅜	4.96	1.21
" ½	6.50	1.19
" ⅝	7.96	1.17
5x3x⅜	5.72	1.16
" ½	7.50	1.16
" ⅝	9.22	1.15
" ¾	10.88	1.15
5x3½x⅜	6.10	1.37
" ½	8.00	1.35
" ⅝	9.84	1.34
" ¾	11.62	1.33
6x4x⅜	7.22	1.56
" ½	9.50	1.56
" ⅝	11.72	1.55
" ¾	13.88	1.55
" ⅞	15.96	1.54
" 1	18.00	1.54
8x6x½	13.50	2.39
" ⅝	16.72	2.38
" ¾	19.88	2.36
" ⅞	22.96	2.35
" 1	26.00	2.34

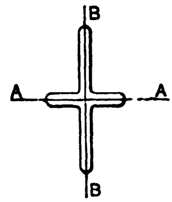
For unequal leg angles, the angle between *B–B* & *C–C* varies between 10° & 34°.
Tie plates for unequal leg angles = ⅜″.

Four Angles Starred, Equal Legs.



Size of Angles.	Total Area.	Radius of Gyration.
	A	r_A
In.	In.²	In.
2x2x¼	3.76	.85
" ⅜	5.44	.88
2½x2½x¼	4.76	1.05
" ⅜	6.92	1.07
3x3x¼	5.76	1.25
" ⅜	8.44	1.27
" ½	11.00	1.29
" ⅝	13.44	1.32
3½x3½x¼	6.76	1.45
" ⅜	9.92	1.48
" ½	13.00	1.50
" ⅝	15.92	1.52
4x4x¼	7.76	1.66
" ⅜	11.44	1.68
" ½	15.00	1.70
" ⅝	18.44	1.72
5x5x⅜	14.44	2.08
" ½	19.00	2.10
" ⅝	23.44	2.12
" ¾	27.76	2.14
6x6x⅜	17.44	2.49
" ½	23.00	2.51
" ⅝	28.44	2.53
" ¾	33.76	2.55
" ⅞	38.92	2.57
" 1	44.00	2.59
8x8x½	31.00	3.32
" ⅝	38.44	3.34
" ¾	45.76	3.36
" ⅞	52.92	3.38
" 1	60.00	3.40

Four Angles Starred, Unequal Legs.



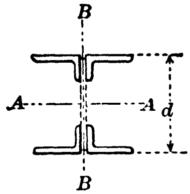
Size of Angles.	Total Area.	Radius of Gyration. Axis A–A.	Axis B–B.
	A	r_A	r_B
In.	In.²	In.	In.
2½x2x¼	4.24	1.11	.80
" ⅜	6.20	1.13	81
3x2½x¼	5.24	1.31	1.00
" ⅜	7.68	1 33	1.02
3½x3x¼	6.24	1.52	1.20
" ⅜	9.20	1.53	1.23
" ½	12.00	1.55	1.24
" ⅝	14.68	1.57	1.26
4x3x¼	6.76	1.77	1.16
" ⅜	9.92	1.80	1.17
" ½	13.00	1.82	1.20
" ⅝	15.92	1.84	1.22
5x3x⅜	11.44	2.34	1.09
" ½	15.00	2.36	1.11
" ⅝	18.44	2.39	1.14
" ¾	21.76	2.41	1.16
5x3½x⅜	12.20	2.27	1.34
" ½	16.00	2.29	1.36
" ⅝	19.68	2.31	1.38
" ¾	23.24	2.33	1.40
6x4x⅜	14.44	2.74	1.50
" ½	19.00	2.76	1.51
" ⅝	23.44	2.78	1.53
" ¾	27.76	2.80	1.56
" ⅞	31.92	2.82	1.58
" 1	36.00	2.85	1.60
8x6x½	27.00	3.56	2.32
" ⅝	33.44	3.58	2.33
" ¾	39.76	3.60	2.35
" ⅞	45.92	3.62	2.37
" 1	52.00	3.64	2.39

When angles are not in contact, use tables 38, 39, & 40.

TABLE 68.

Properties of Four Angles Laced.

Properties of Four Angles Laced.



For Equal Legs and Unequal Legs with Long Legs Turned Out.

Four Angles.	Total Area.	Moments of Inertia and Radii of Gyration.													
		Axis B-B.				Axis A-A.									
		Thickness of 2 Lacing Bars = t.				Distance Back to Back of Angles in Inches = d.									
		2 Bars ¼″ = ½″.		2 Bars 5/16″ = ⅝″.		8½		10½		12½		14½		16½	
		I_B	r_B	I_B	r_B	I_A	r_A	I_A	r_A	I_A	r_A	I_A	r_A	I_A	r_A
In.	In.²	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.⁴	In.	In.⁴	In.
3x2½x¼	5.24	12	1.50	13	1.55	71	3.68	113	4.64	167	5.64	231	6.64	305	7.63
" ⅜	7.68	18	1.53	19	1.58	100	3.61	162	4.59	240	5.59	333	6.58	440	7.58
" ½	10.00	24	1.55	26	1.60	128	3.57	208	4.56	308	5.55	428	6.54	567	7.54
4x3x⅜	9.92	39	1.98	41	2.03	127	3.58	206	4.56	305	5.55	423	6.53	561	7.52
" ½	13.00	53	2.01	55	2.06	162	3.53	264	4.51	392	5.49	546	6.48	725	7.48
" ⅝	15.92	66	2.04	69	2.08	193	3.48	317	4.46	472	5.44	659	6.43	879	7.42
		2 Bars ¼″ = ½″		2 Bars 5/16″ = ⅝″		10½		12½		14½		16½		18½	
3½x3½x⅜	9.92	27	1.66	29	1.71	190	4.38	284	5.34	398	6.34	532	7.32	685	8.31
" ½	13.00	37	1.69	39	1.73	243	4.32	365	5.30	513	6.28	687	7.27	887	8.26
" ⅝	15.92	46	1.70	49	1.76	291	4.27	440	5.26	619	6.23	831	7.18	1075	8.21
4x4x⅜	11.44	39	1.86	42	1.91	211	4.29	316	5.25	444	6.22	596	7.22	770	8.20
" ½	15.00	53	1.88	56	1.93	271	4.25	408	5.22	575	6.19	772	7.17	999	8.16
" ⅝	18.44	67	1.91	71	1.96	325	4.20	491	5.16	695	6.14	935	7.12	1213	8.11
		2 Bars 5/16″ = ⅝″		2 Bars ⅜″ = ¾″		10½		12½		14½		16½		18½	
5x3½x⅜	12.20	76	2.50	79	2.55	248	4.51	367	5.48	511	6.47	679	7.46	872	8.45
" ½	16.00	102	2.53	106	2.58	318	4.46	472	5.43	659	6.41	878	7.41	1129	8.40
" ⅝	19.68	128	2.55	133	2.60	382	4.40	571	5.39	800	6.37	1067	7.36	1374	8.36
6x4x½	19.00	170	2.99	176	3.04	370	4.41	551	5.39	770	6.36	1027	7.35	1321	8.34
" ⅝	23.44	213	3.01	220	3.06	448	4.37	669	5.34	937	6.32	1252	7.32	1614	8.30
" ¾	27.76	257	3.04	265	3.09	517	4.32	777	5.29	1092	6.27	1462	7.26	1888	8.24

The above table is intended to serve only as a guide in the choice of sections and not as a complete table. The properties of other sections may be found as follows:

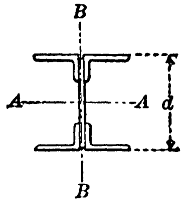
The areas and moments of inertia of four angles about the axis *A–A* are given in Table 32, for equal leg angles; Table 33, for unequal leg angles, long legs out, and Table 34, unequal leg angles, short legs out; the axis *A–A* corresponding to axis *X–X* in Tables. The radius of gyration about axis *A–A* may be calculated from the formula $r_A = \sqrt{I_A \div A}$.

The moments of inertia of four angles about the axis *B–B* are given in Tables 35, 36 and 37, the axis *B–B* corresponding to *Y–Y* in Tables. The radii of gyration of four angles about the axis *B–B* may be calculated from the formula $r_B = \sqrt{I_B \div A}$, or may be found from Tables 38, 39 and 40, the radius of gyration of four angles being equal to that of two angles.

TABLE 69.

Properties of Four Angles and One Plate.

Properties of Plate and Angle Column Sections.



Without Flange Plates Long Legs Out.
d = Width of Web Plate Plus ½ In.

Series I and II.	Series I.						Series II.					
Web Plate.	Four Angles.	Total Area.	Moments of Inertia and Radii of Gyration. Axis A-A. I_A	Axis A-A. r_A	Axis B-B. I_B	Axis B-B. r_B	Four Angles.	Total Area.	Moments of Inertia and Radii of Gyration. Axis A-A. I_A	Axis A-A. r_A	Axis B-B. I_B	Axis B-B. r_B
In.	In.	In.²	In.⁴	In	In.⁴	In.	In.	In.²	In.⁴	In.	In.⁴	In
8x¼	3x2½x¼	7.24	81	3.36	10	1.19	3½x2½x¼	7.76	90	3.41	16	1.44
"	" 5/16	8.48	97	3.38	13	1.23	" 5/16	9.12	108	3.43	20	1.49
8x5/16	3½x2½x5/16	9.62	110	3.38	21	1.47	4x3x5/16	10.86	122	3.35	30	1.67
"	" 3/8	10.94	127	3.40	25	1.51	" 3/8	12.42	141	3.36	36	1.71
8x3/8	4x3x3/8	12.92	143	3.33	37	1.70	4x3x½	16.00	178	3.33	50	1.77
"	" 7/16	14.48	161	3.34	43	1.73	" 9/16	17.48	194	3.33	56	1.79
10x5/16	3½x2½x5/16	10.25	181	4.20	21	1.42	4x3x5/16	11.49	201	4.18	30	1.62
"	" 3/8	11.57	208	4.24	25	1.47	" 3/8	13.05	232	4.22	36	1.67
10x3/8	4x3x3/8	13.67	237	4.16	37	1.65	6x4x3/8	18.19	319	4.19	119	2.56
"	" 7/16	15.23	267	4.18	44	1.69	" 7/16	20.47	361	4.20	139	2.61
"	5x3½x3/8	15.95	279	4.18	71	2.10	" ½	22.75	401	4.20	160	2.65
"	" 7/16	17.87	315	4.20	82	2.15	" 9/16	24.99	440	4.19	180	2.69
10x½	5x3½x½	21.00	360	4.14	98	2.16	6x4x½	24.00	412	4.14	165	2.62
"	" 9/16	22.88	393	4.14	111	2.20	" 9/16	26.24	451	4.15	187	2.66
"	" 5/8	24.68	424	4.15	123	2.22	" 5/8	28.44	489	4.15	206	2.69
12x5/16	4x3x5/16	12.11	304	5.01	30	1.57	5x3½x5/16	13.99	355	5.02	58	2.04
"	" 3/8	13.67	350	5.06	36	1.62	" 3/8	15.95	412	5.04	69	2.08
12x3/8	4x3x3/8	14.42	359	4.99	37	1.60	6x4x3/8	18.94	481	5.04	119	2.51
"	" 7/16	15.98	404	5.02	44	1.66	" 7/16	21.22	544	5.06	139	2.56
"	5x3½x3/8	16.70	421	5.02	70	2.05	" ½	23.50	605	5.07	160	2.61
"	" 7/16	18.62	476	5.04	82	2.10	" 9/16	25.74	665	5.08	180	2.65
"	" ½	20.50	526	5.06	95	2.15	" 5/8	27.94	723	5.09	200	2.67
12x½	5x3½x½	22.00	544	4.97	98	2.11	6x4x½	25.00	623	4.99	165	2.57
"	" 9/16	23.88	596	5.00	111	2.16	" 9/16	27.24	683	5.01	186	2.61
"	" 5/8	25.68	643	5.00	123	2.19	" 5/8	29.44	741	5.02	206	2.65
"	" 11/16	27.48	692	5.02	135	2.21	" 11/16	31.60	794	5.01	228	2.69
"	" ¾	29.24	735	5.01	149	2.26	" ¾	33.76	849	5.01	249	2.72

The above table is intended to serve only as a guide in the choice of sections and not as a complete table. The properties of other sections may be found as follows:

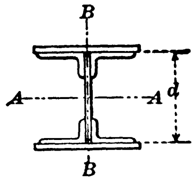
Example: Required the properties of a section composed of 4 ∠ 5″ × 3½″ × 7/16″, long legs out, 12¼″ back to back, and one plate 12″ × 7/16″.

Item.	Area.		Moment of Inertia. Axis A-A.		Axis B-B.		Radius of Gyration. Axis A-A.	Axis B-B.
	Table No.	A	Table No.	I_A	Table No.	I_B	$r_A = \sqrt{I_A \div A}$	$r_B = \sqrt{I_B \div A}$
In.		In.²		In.⁴		In.⁴	In.	In.
4∠5x3½x7/16	33	14.12	33	403	36	84	$\sqrt{\frac{466}{19.37}}$	$\sqrt{\frac{84}{19.37}}$
1 Pl—12x7/16	1	5.25	3	63	4	0		
Totals	A =	19.37	I_A =	466	I_B =	84	r_A = 4.90	r_B = 2.08

TABLE 70.

PROPERTIES OF FOUR ANGLES AND THREE PLATES.

Properties of Plate and Angle Column Sections.



With Flange Plates. d = Width of Web Plate Plus ¼ In.

Series I and II.		Series I.						Series II.					
Web Plate.	Four Angles.	Two Cover Plates.	Total Area.	Moments of Inertia and Radii of Gyration. Axis A-A. I_A	r_A	Axis B-B. I_B	r_B	Two Cover Plates.	Total Area.	Moments of Inertia and Radii of Gyration. Axis A-A. I_A	r_A	Axis B-B. I_B	r_B
In.	In.	In.	In.^{2}	In.^{4}	In.	In.^{4}	In.	In.	In.^{2}	In.^{4}	In.	In.^{4}	In.
10x⅜	4x3x⅜	10x⅜	21.17	459	4.62	100	2.17	10x½	23.67	540	4.78	121	2.26
"	" ½	" ½	26.75	598	4.73	134	2.24	" ⅝	29.25	682	5.16	154	2.46
10x½	5x3½x⅜	12x⅜	26.20	556	4.60	181	2.63	12x½	29.20	653	4.73	217	2.73
"	" ½	" ½	33.00	723	4.68	242	2.71	" ⅝	36.00	824	4.78	278	2.78
12x⅜	5x3½x⅜	12x⅜	25.70	794	5.31	179	2.64	12x½	28.70	929	5.69	215	2.74
"	" ½	" ½	32.50	1034	5.66	239	2.71	" ⅝	35.50	1173	5.75	275	2.78
12x½	5x3½x½	" ½	34.00	1052	5.59	242	2.67	" ⅝	37.00	1191	5.68	278	2.74
"	" ⅝	" ⅝	40.68	1290	5.63	303	2.73	" ¾	43.68	1387	5.64	339	2.78
12x⅜	6x4x⅜	14x⅜	29.44	916	5.58	291	3.14	14x½	32.94	1073	5.71	348	3.25
"	" ½	" ½	37.50	1197	5.65	388	3.22	" ⅝	41.00	1360	5.76	446	3.29
12x½	6x4x½	" ½	39.00	1215	5.58	394	3.18	" ⅝	42.50	1378	5.69	451	3.26
"	" ⅝	" ⅝	46.94	1496	5.64	492	3.24	" ¾	50.44	1664	5.75	549	3.30
14x⅜	6x4x⅜	14x⅜	30.19	1261	6.46	291	3.10	14x½	33.69	1469	6.60	348	3.21
"	" ½	" ½	38.25	1644	6.55	388	3.19	" ⅝	41.75	1857	6.67	446	3.27
14x½	6x4x½	" ½	40.00	1672	6.46	394	3.14	" ⅝	43.50	1885	6.58	451	3.22
"	" ⅝	" ⅝	47.94	2052	6.54	492	3.20	" ¾	51.44	2263	6.63	549	3.26
14x⅝	6x4x⅝	" ⅝	49.69	2081	6.47	499	3.17	" ¾	53.19	2292	6.57	556	3.23
"	"	" ⅞	56.69	2529	6.68	613	3.29	" 1	60.19	2764	6.74	671	3.34
"	"	" 1⅛	63.69	3006	6.87	728	3.38	" 1¼	67.19	3255	6.96	785	3.42
"	"	" 1⅜	70.69	3512	7.05	842	3.45	" 1½	74.19	3776	7.13	899	3.48
"	"	" 1⅝	77.69	4048	7.22	956	3.51	" 1¾	81.19	4327	7.30	1014	3.53
"	"	" 1⅞	84.69	4615	7.38	1071	3.56	" 2	88.19	4910	7.46	1128	3.58
"	"	" 2⅛	91.69	5214	7.54	1185	3.60	" 2¼	95.19	5525	7.62	1242	3.62
"	"	" 2⅜	98.69	5846	7.69	1299	3.63	" 2½	102.19	6175	7.77	1356	3.64

The above table is intended to serve only as a guide in the choice of sections and not as a complete table. The properties of other sections may be found as follows:

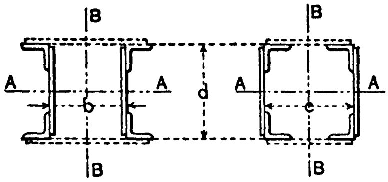
Example: Required the properties of a section composed of 4 ∠ 5″ × 3½″ × $\frac{7}{16}$″, long legs out, 12¼″ back to back, one web plate 12″ × $\frac{7}{16}$″ and two flange plates 12″ × ⅜″.

Item.	Area.		Moment of Inertia.				Radius of Gyration.	
			Axis A-A.		Axis B-B.		Axis A-A.	Axis B-B.
	Table No.	A	Table No.	I_A	Table No.	I_B	$r_A = \sqrt{I_A \div A}$	$r_B = \sqrt{I_B \div A}$
In.		In.^{2}		In.^{4}		In.^{4}	In.	In.
4∠5x3½x$\frac{7}{16}$	33	14.12	33	403	36	84	$\sqrt{\frac{825}{28.37}}$	$\sqrt{\frac{192}{28.37}}$
1 Pl—12x$\frac{7}{16}$	1	5.25	3	63	4	0		
2 Pl—12x⅜	1	9.00	5	359	3	108		
Total	A =	28.37	I_A =	825	I_B =	192	r_A = 5.39	r_B = 2.60

TABLE 71.

Properties of Four Angles and Two Plates, Laced.

Properties of Four Angles and Two Plates, Laced. Angles Turned Out and Angles Turned In.



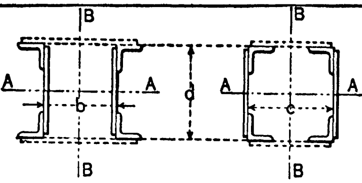
b = Width, Back to Back of Angles, for Equal Moments of Inertia about Axes A–A and B–B with Angles Turned Out.
c = Same as b, but with Angles Turned in.
d = Depth of Web Plates + ½″.

Series 1, 2, 3 and 4. Size of Angles.	Series 1. Total Area.	Series 1. Moment of Inertia.	Series 1. Radius of Gyration.	Series 1. b. to b. Angles.		Series 2. Total Area.	Series 2. Moment of Inertia.	Series 2. Radius of Gyration.	Series 2. b. to b. Angles.		Series 3. Total Area.	Series 3. Moment of Inertia.	Series 3. Radius of Gyration.	Series 3. b. to b. Angles.		Series 4. Total Area.	Series 4. Moment of Inertia.	Series 4. Radius of Gyration.	Series 4. b. to b. Angles.	
	A	I	r	b	c	A	I	r	b	c	A	I	r	b	c	A	I	r	b	c
In.	In.²	In.⁴	In.	In.	In.	In.²	In.⁴	In.	In.	In.	In.²	In.⁴	In.	In.	In.	In.²	In.⁴	In.	In.	In.
	8″x¼″ Web Plates.					8″x⅜″ Web Plates.					8″x½″ Web Plates.					8″x⅝″ Web Plates.				
2½x2½x¼	8.76	83	3.08	5.4	6.7	10.76	94	2.95	5.3	6.3	12.76	105	2.87	5.4	5.8	14.76	115	2.79	5.3	5.4
" ⅜	10.92	109	3.16	5.3	7.0	12.92	119	3.04	5.3	6.6	14.92	130	2.95	5.4	6.1	16.92	141	2.89	5.2	5.8
" ½	13.00	132	3.19	5.2	7.3	15.00	143	3.09	5.2	6.9	17.00	154	3.01	5.3	6.5	19.00	165	2.95	5.2	6.1
3x3x¼	9.76	93	3.09	5.1	6.8	11.76	104	2.97	5.1	6.4	13.76	115	2.89	5.1	6.0	15.76	126	2.83	5.1	5.6
" ⅜	12.44	123	3.15	5.0	7.1	14.44	134	3.05	5.0	6.7	16.44	145	2.97	5.0	6.4	18.44	156	2.91	5.0	6.0
" ½	15.00	151	3.17	4.8	7.4	17.00	162	3.09	4.9	7.0	19.00	173	3.02	4.9	6.7	21.00	184	2.96	5.0	6.3
3½x3½x⅜	13.92	137	3.14	4.6	7.3	15.92	148	3.05	4.7	6.9	17.92	159	2.98	4.7	6.6	19.92	170	2.92	4.8	6.2
" ½	17.00	168	3.15	4.5	7.5	19.00	179	3.07	4.6	7.2	21.00	190	3.01	4.5	6.9	23.00	201	2.96	4.6	6.5
" ⅝	19.92	196	3.15	4.3	7.7	21.92	207	3.08	4.4	7.4	23.92	218	3.02	4.3	7.1	25.92	229	2.97	4.4	6.8
	10″x¼″ Web Plates.					10″x⅜″ Web Plates.					10″x½″ Web Plates.					10″x⅝″ Web Plates.				
2½x2½x¼	9.76	142	3.82	6.4	7.5	12.26	162	3.63	6.5	7.3	14.76	183	3.52	6.6	7.0	17.26	204	3.44	6.8	6.7
" ⅜	11.92	185	3.94	6.6	8.1	14.42	205	3.77	6.7	7.8	16.92	226	3.66	6.7	7.5	19.42	247	3.56	6.8	7.1
" ½	14.00	224	4.00	6.9	8.8	16.50	244	3.85	6.9	8.4	19.00	265	3.73	6.8	8.0	21.50	286	3.65	6.8	7.5
3x3x¼	10.76	159	3.84	6.7	8.3	13.26	179	3.68	6.7	7.8	15.76	200	3.56	6.7	7.4	18.26	221	3.48	6.7	6.9
" ⅜	13.44	209	3.94	6.7	8.7	15.94	229	3.79	6.7	8.2	18.44	250	3.67	6.6	7.8	20.94	271	3.60	6.6	7.3
" ½	16.00	256	4.00	6.6	9.0	18.50	276	3.86	6.6	8.6	21.00	297	3.76	6.6	8.1	23.50	318	3.68	6.6	7.7
3½x3½x⅜	14.92	232	3.94	6.4	8.9	17.42	252	3.80	6.5	8.5	19.92	273	3.70	6.4	8.0	22.42	294	3.62	6.4	7.5
" ½	18.00	285	3.98	6.2	9.1	20.50	305	3.86	6.3	8.7	23.00	326	3.76	6.3	8.3	25.50	347	3.69	6.3	7.8
" ⅝	20.92	333	3.99	6.0	9.3	23.42	353	3.88	6.1	8.9	25.92	374	3.78	6.2	8.5	28.42	395	3.72	6.2	8.1
	12″x¼″ Web Plates.					12″x⅜″ Web Plates.					12″x½″ Web Plates.					12″x⅝″ Web Plates.				
2½x2½x¼	10.76	220	4.52	8.4	9.4	13.76	256	4.32	8.3	9.0	16.76	292	4.17	8.2	8.5	19.76	328	4.08	8.2	8.0
" ⅜	12.92	288	4.72	8.5	9.9	15.92	324	4.51	8.4	9.4	18.92	360	4.36	8.3	8.9	21.92	396	4.25	8.3	8.4
" ½	15.00	343	4.78	8.6	10.3	18.00	379	4.59	8.5	9.8	21.00	415	4.45	8.4	9.3	24.00	451	4.34	8.3	8.8
3x3x¼	11.76	246	4.57	8.3	9.7	14.76	282	4.37	8.2	9.3	17.76	318	4.23	8.1	8.8	20.76	354	4.13	8.0	8.3
" ⅜	14.44	322	4.72	8.2	10.2	17.44	358	4.53	8.2	9.7	20.44	394	4.39	8.1	9.2	23.44	430	4.28	8.1	8.7
" ½	17.00	392	4.80	8.2	10.6	20.00	428	4.63	8.2	10.1	23.00	464	4.49	8.2	9.6	26.00	500	4.39	8.2	9.0
3½x3½x⅜	15.92	356	4.73	8.0	10.4	18.92	392	4.55	7.9	9.9	21.92	428	4.42	7.9	9.4	24.92	464	4.31	8.0	8.9
" ½	19.00	437	4.80	8.0	10.7	22.00	473	4.64	7.9	10.2	25.00	509	4.51	7.9	9.7	28.00	545	4.41	8.0	9.2
" ⅝	21.92	512	4.83	7.9	11.0	24.92	548	4.69	7.9	10.6	27.92	584	4.57	7.9	10.1	30.92	620	4.48	7.9	9.6
4x4x⅜	17.44	388	4.72	7.7	10.5	20.44	424	4.58	7.7	10.0	23.44	460	4.43	7.7	9.4	26.44	496	4.33	7.7	9.0
" ½	21.00	480	4.78	7.7	10.8	24.00	516	4.64	7.6	10.3	27.00	552	4.53	7.6	9.8	30.00	588	4.43	7.6	9.3
" ⅝	24.44	563	4.80	7.6	11.1	27.44	599	4.67	7.5	10.6	30.44	635	4.57	7.5	10.1	33.44	671	4.51	7.5	9.7

TABLE 71.—*Continued.*

PROPERTIES OF FOUR ANGLES AND TWO PLATES, LACED.

Properties of Four Angles and Two Plates, Laced. Angles Turned Out and Angles Turned In.



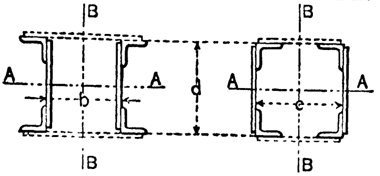
b = Width, Back to Back of Angles, for Equal Moments of Inertia about Axes A–A and B–B when Angles Are Turned Out.
c = Same as b with Angles Turned In.
d = Depth of Web Plates + ½″.

Series 1, 2, 3 and 4. Size of Angles.	Series 1. Total Area.	Moment of Inertia.	Radius of Gyration.	b. to b. Angles.		Series 2. Total Area.	Moment of Inertia.	Radius of Gyration.	b. to b. Angles.		Series 3. Total Area.	Moment of Inertia.	Radius of Gyration.	b. to b. Angles.		Series 4. Total Area.	Moment of Inertia.	Radius of Gyration.	b. to b. Angles.	
	A	I	r	b	c	A	I	r	b	c	A	I	r	b	c	A	I	r	b	c
In.	In.²	In.⁴	In.	In.	In.	In.²	In.⁴	In.	In.	In.	In.²	In.⁴	In.	In.	In.	In.²	In.⁴	In.	In.	In.
	14″ x ⅜″ Web Plates.					14″ x ½″ Web Plates.					14″ x ⅝″ Web Plates.					14″ x ¾″ Web Plates.				
3x3x¼	16.26	414	5.05	9.6	10.3	19.76	471	4.89	9.6	10.0	23.26	528	4.77	9.5	9.5	26.76	585	4.67	9.6	9.0
" ⅜	18.94	520	5.24	9.7	10.9	22.44	577	5.07	9.7	10.4	25.94	634	4.94	9.6	9.9	29.44	691	4.84	9.6	9.5
" ½	21.50	620	5.37	9.8	11.4	25.00	677	5.20	9.8	10.8	28.50	734	5.07	9.7	10.3	32.00	791	4.97	9.6	10.0
3½x3½x⅜	20.42	570	5.28	9.6	11.1	23.92	627	5.12	9.6	10.6	27.42	684	4.99	9.5	10.2	30.92	741	4.89	9.5	9.8
" ½	23.50	685	5.40	9.6	11.6	27.00	742	5.25	9.6	11.1	30.50	799	5.12	9.5	10.6	34.00	856	5.02	9.5	10.1
" ⅝	26.42	791	5.47	9.6	12.1	29.92	848	5.32	9.6	11.6	33.42	905	5.20	9.5	11.0	36.92	962	5.10	9.5	10.5
4x4x⅜	21.94	616	5.30	9.3	11.4	25.44	673	5.15	9.3	10.9	28.94	730	5.02	9.4	10.5	32.44	787	4.93	9.4	10.0
" ½	25.50	747	5.41	9.3	11.8	29.00	804	5.26	9.3	11.3	32.50	861	5.15	9.3	10.8	36.00	918	5.05	9.4	10.4
" ⅝	28.94	867	5.47	9.2	12.1	32.44	924	5.34	9.2	11.7	35.94	981	5.23	9.3	11.2	39.44	1038	5.13	9.3	10.8
	16″ x ½″ Web Plates.					16″ x ⅝″ Web Plates.					16″ x ¾″ Web Plates.					16″ x ⅞″ Web Plates.				
3½x3½x⅜	25.92	873	5.80	11.0	12.0	29.92	959	5.66	11.0	11.5	33.92	1044	5.53	10.9	11.0	37.92	1129	5.46	10.9	10.5
" ½	29.00	1028	5.96	11.1	12.4	33.00	1114	5.81	11.0	11.9	37.00	1199	5.69	11.0	11.5	41.00	1284	5.60	10.9	11.0
" ⅝	31.92	1172	6.06	11.1	12.8	35.92	1258	5.92	11.1	12.3	39.92	1343	5.80	11.0	11.9	43.92	1428	5.70	11.0	11.5
4x4x⅜	27.44	937	5.84	10.9	12.1	31.44	1023	5.71	10.9	11.7	35.44	1108	5.60	10.9	11.5	39.44	1193	5.50	10.8	11.1
" ½	31.00	1113	5.99	10.9	12.5	35.00	1199	5.85	10.9	12.2	39.00	1284	5.74	10.9	11.8	43.00	1369	5.64	10.8	11.4
" ⅝	34.44	1276	6.09	10.9	13.0	38.44	1362	5.96	10.9	12.6	42.44	1447	5.84	10.8	12.1	46.44	1532	5.74	10.8	11.7
6x6x⅜	33.44	1165	5.90	9.8	12.8	37.44	1251	5.78	9.8	12.4	41.44	1336	5.68	9.9	12.1	45.44	1421	5.60	10.2	11.6
" ½	39.00	1413	6.02	9.7	13.2	43.00	1499	5.91	9.7	12.8	47.00	1584	5.81	9.8	12.6	51.00	1669	5.72	10.1	12.1
" ⅝	44.44	1647	6.09	9.6	13.6	48.44	1733	5.98	9.6	13.2	52.44	1818	5.89	9.7	13.0	56.44	1903	5.81	10.0	12.5
" ¾	49.76	1867	6.12	9.5	14.0	53.76	1953	6.03	9.5	13.6	57.76	2038	5.94	9.6	13.4	61.76	2123	5.87	9.9	12.9
	18″ x ½″ Web Plates.					18″ x ⅝″ Web Plates.					18″ x ¾″ Web Plates.					18″ x ⅞″ Web Plates.				
3½x3½x⅜	27.92	1171	6.49	12.4	13.2	32.42	1293	6.32	12.4	12.8	36.92	1414	6.19	12.5	12.5	41.42	1536	6.09	12.4	12.1
" ½	31.00	1373	6.66	12.6	13.7	35.50	1495	6.49	12.5	13.3	40.00	1616	6.36	12.5	12.9	44.50	1738	6.25	12.4	12.4
" ⅝	33.92	1561	6.78	12.7	14.2	38.42	1683	6.61	12.6	13.7	42.92	1804	6.48	12.5	13.2	47.42	1926	6.38	12.4	12.7
4x4x⅜	29.44	1256	6.53	12.4	13.5	33.94	1378	6.38	12.2	12.9	38.44	1499	6.25	12.2	12.6	42.94	1621	6.14	12.1	12.1
" ½	33.00	1485	6.71	12.5	14.0	37.50	1607	6.55	12.3	13.4	42.00	1728	6.42	12.3	13.0	46.50	1850	6.31	12.2	12.5
" ⅝	36.44	1699	6.82	12.6	14.5	40.94	1821	6.67	12.4	13.9	45.44	1942	6.54	12.4	13.4	49.94	2064	6.43	12.3	12.9
6x6x½	41.00	1884	6.78	11.5	14.8	45.50	2006	6.64	11.5	14.3	50.00	2127	6.53	11.5	13.8	54.50	2249	6.43	11.4	13.3
" ⅝	46.44	2191	6.87	11.3	15.2	50.94	2313	6.74	11.3	14.7	55.44	2434	6.63	11.3	14.2	59.94	2556	6.53	11.4	13.7
" ¾	51.76	2482	6.92	11.2	15.3	56.26	2604	6.80	11.2	15.1	60.76	2725	6.69	11.3	14.6	65.26	2847	6.59	11.3	14.1
" ⅞	56.92	2762	6.96	11.1	15.2	61.42	2884	6.85	11.1	15.5	65.92	3005	6.74	11.2	15.0	70.42	3127	6.66	11.2	14.5

TABLE 71.—*Continued.*

Properties of Four Angles and Two Plates, Laced.

Properties of Four Angles and Two Plates, Laced. Angles Turned Out and Angles Turned In.



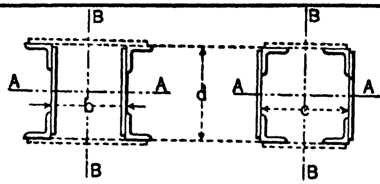
b = Width, Back to Back of Angles for Equal Moments of Inertia about Axes A–A and B–B with Angles Turned Out.
c = Same as b, but with Angles Turned In.
d = Depth of Web Plates + ½″.

Series 1, 2, 3 and 4. Size of Angles.	Series 1. Total Area.	Series 1. Moment of Inertia.	Series 1. Radius of Gyration.	Series 1. b. to b. Angles.		Series 2. Total Area.	Series 2. Moment of Inertia.	Series 2. Radius of Gyration.	Series 2. b. to b. Angles.		Series 3. Total Area.	Series 3. Moment of Inertia.	Series 3. Radius of Gyration.	Series 3. b. to b. Angles.		Series 4. Total Area.	Series 4. Radius of Gyration.	Series 4. b. to b. Angles.	
	A	I	r	b	c	A	I	r	b	c	A	I	r	b	c	A	r	b	c
In.	In.²	In.⁴	In.	In.	In.	In.²	In.⁴	In.	In.	In.	In.²	In.⁴	In.	In.	In.	In.²	In.	In.	In.
	20″x½″ Web Plates.					20″x⅝″ Web Plates.					20″x¾″ Web Plates.					20″x⅞″ Web Plates.			
3½x3½x⅜	29.92	1525	7.14	13.8	14.5	34.92	1691	6.96	13.7	14.0	39.92	1858	6.83	13.6	13.5	44.92	6.72	13.5	13.0
" ½	33.00	1779	7.34	14.0	15.0	38.00	1945	7.15	13.9	14.5	43.00	2112	7.02	13.8	14.0	48.00	6.90	13.6	13.5
" ⅝	35.92	2017	7.50	14.2	15.6	40.92	2183	7.31	14.0	15.0	45.92	2350	7.15	13.9	14.5	50.92	7.03	13.7	14.0
4x4x⅜	31.44	1634	7.21	13.7	14.8	36.44	1800	7.03	13.6	14.2	41.44	1967	6.89	13.6	13.8	46.44	6.78	13.5	13.3
" ½	35.00	1923	7.41	13.9	15.4	40.00	2089	7.23	13.8	14.8	45.00	2256	7.08	13.7	14.3	50.00	6.96	13.6	13.8
" ⅝	38.44	2194	7.58	14.1	16.0	43.44	2360	7.37	13.9	15.3	48.44	2527	7.23	13.9	14.8	53.44	7.10	13.7	14.2
6x6x½	43.00	2436	7.53	13.1	16.2	48.00	2602	7.36	13.2	15.6	53.00	2769	7.23	13.3	15.2	58.00	7.12	13.4	14.2
" ⅝	48.44	2828	7.64	13.1	16.6	53.44	2994	7.49	13.1	16.1	58.44	3161	7.36	13.2	15.6	63.44	7.24	13.3	14.7
" ¾	53.76	3202	7.72	13.0	17.0	58.76	3368	7.57	13.0	16.5	63.76	3535	7.45	13.1	16.0	68.76	7.34	13.1	15.2
" ⅞	58.92	3561	7.79	12.9	17.4	63.92	3727	7.64	12.9	16.9	68.92	3894	7.52	12.9	16.4	73.92	7.42	12.9	15.7
	22″x⅝″ Web Plates.					22″x¾″ Web Plates.					22″x⅞″ Web Plates.					22″x1″ Web Plates.			
3½x3½x⅜	37.42	2161	7.60	15.0	15.2	42.92	2383	7.45	14.9	14.8	48.42	2605	7.34	14.9	14.3	53.92	7.24	14.8	13.9
" ½	40.50	2473	7.82	15.3	15.7	46.00	2695	7.68	15.2	15.3	51.50	2917	7.53	15.1	14.8	57.00	7.43	15.0	14.4
" ⅝	43.42	2766	7.98	15.5	16.2	48.92	2988	7.82	15.4	15.8	54.42	3210	7.67	15.3	15.3	59.92	7.57	15.2	14.9
4x4x⅜	38.94	2296	7.68	15.0	15.5	44.44	2518	7.54	15.0	15.2	49.94	2740	7.41	15.1	14.8	55.44	7.30	15.1	14.2
" ½	42.50	2652	7.90	15.3	16.1	48.00	2874	7.74	15.2	15.7	53.50	3096	7.61	15.2	15.3	59.00	7.51	15.1	14.7
" ⅝	45.94	2988	8.07	15.6	16.7	51.44	3210	7.90	15.4	16.2	56.94	3432	7.76	15.3	15.7	62.44	7.65	15.1	15.3
6x6x½	50.50	3295	8.08	14.6	17.0	56.00	3517	7.93	14.6	16.5	61.50	3739	7.80	14.6	16.1	67.00	7.69	14.6	15.6
" ⅝	55.94	3783	8.22	14.6	17.4	61.44	4005	8.08	14.6	16.9	66.94	4227	7.93	14.6	16.5	72.44	7.83	14.6	16.0
" ¾	61.26	4249	8.33	14.6	17.9	66.76	4471	8.19	14.6	17.4	72.26	4693	8.05	14.6	16.9	77.76	7.96	14.6	16.5
" ⅞	66.42	4698	8.42	14.6	18.3	71.92	4920	8.27	14.6	17.8	77.42	5142	8.15	14.6	17.4	82.92	8.04	14.5	16.9
	24″x⅝″ Web Plates.					24″x¾″ Web Plates.					24″x⅞″ Web Plates.					24″x1″ Web Plates.			
4x4x⅜	41.44	2870	8.32	16.4	16.7	47.44	3158	8.16	16.3	16.3	53.44	3446	8.03	16.1	16.0	59.44	7.93	16.0	15.6
" ½	45.00	3300	8.56	16.6	17.3	51.00	3588	8.47	16.5	16.9	57.00	3876	8.25	16.4	16.5	63.00	8.14	16.3	16.0
" ⅝	48.44	3707	8.75	16.8	17.9	54.44	3995	8.57	16.7	17.4	60.44	4283	8.42	16.6	16.9	66.44	8.30	16.5	16.4
6x6x½	53.00	4089	8.79	16.2	18.4	59.00	4377	8.62	16.1	17.9	65.00	4665	8.47	16.0	17.4	71.00	8.36	16.0	16.9
" ⅝	58.44	4684	8.96	16.2	18.9	64.44	4972	8.79	16.1	18.4	70.44	5260	8.64	16.0	17.9	76.44	8.53	16.0	17.4
" ¾	63.76	5253	9.08	16.2	19.3	69.76	5541	8.92	16.2	18.9	75.76	5829	8.77	16.1	18.3	81.76	8.66	16.1	17.8
" ⅞	68.92	5802	9.18	16.2	19.8	74.92	6090	9.02	16.2	19.3	80.92	6378	8.88	16.1	18.8	86.92	8.76	16.1	18.3
8x8x½	61.00	4772	8.85	15.3	19.0	67.00	5060	8.69	15.3	18.5	73.00	5348	8.56	15.3	18.0	79.00	8.45	15.3	17.5
" ⅝	68.44	5537	8.98	15.2	19.6	74.44	5825	8.85	15.2	19.1	80.44	6113	8.72	15.2	18.6	86.44	8.60	15.3	18.0
" ¾	75.76	6268	9.11	15.1	20.1	81.76	6556	8.96	15.1	19.6	87.76	6844	8.84	15.1	19.1	93.76	8.72	15.3	18.5
" ⅞	82.92	6976	9.16	15.0	20.5	88.92	7264	9.04	15.0	19.9	94.92	7552	8.93	15.0	19.4	100.92	8.82	15.2	19.0
" 1	90.00	7653	9.22	14.9	20.8	96.00	7941	9.10	14.9	20.2	102.00	8229	8.99	14.9	19.7	108.00	8.89	15.2	19.5

TABLE 71.—*Continued.*

PROPERTIES OF FOUR ANGLES AND TWO PLATES, LACED.

Properties of Four Angles and Two Plates, Laced. Angles Turned Out and Angles Turned In.



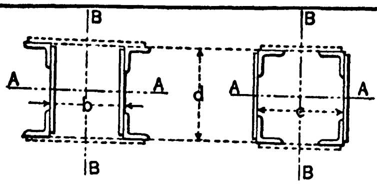
b = Width, Back to Back of Angles, for Equal Moments of Inertia about Axes A–A and B–B for Angles Turned Out.
c = Same as b, but with Angles Turned In.
d = Depth of Web Plates + ½″.

Series 1, 2, 3 and 4. Size of Angles.	Series 1. Total Area.	Series 1. Moment of Inertia.	Series 1. Radius of Gyration.	Series 1. b. to b. Angles.		Series 2. Total Area.	Series 2. Moment of Inertia.	Series 2. Radius of Gyration.	Series 2. b. to b. Angles.		Series 3. Total Area.	Series 3. Radius of Gyration.	Series 3. b. to b. Angles.		Series 4. Total Area.	Series 4. Radius of Gyration.	Series 4. b. to b. Angles.	
	A	I	r	b	c	A	I	r	b	c	A	r	b	c	A	r	b	c
In.	In.²	In.⁴	In.	In.	In	In.²	In.⁴	In.	In.	In.	In.²	In.	In.	In.	In.²	In.	In.	In.
	26″ x ⅝″ Web Plates.					26″ x ¾″ Web Plates.					26″ x ⅞″ Web Plates.				26″ x 1″ Web Plates.			
4x4x⅜	43.94	3526	8.96	17.7	18.0	50.44	3892	8.79	17.6	17.6	56.94	8.63	17.5	17.1	63.44	8.54	17.4	16.6
" ½	47.50	4039	9.23	18.0	18.6	54.00	4405	9.05	17.8	18.1	60.50	8.88	17.7	17.6	67.00	8.76	17.6	17.1
" ⅝	50.94	4523	9.42	18.2	19.2	57.44	4889	9.23	18.1	18.7	63.94	9.07	18.0	18.2	70.44	8.94	17.9	17.7
6x6x½	55.50	4990	9.48	17.7	19.7	62.00	5356	9.29	17.7	19.2	68.50	9.15	17.6	18.7	75.00	9.02	17.5	18.1
" ⅝	60.94	5702	9.68	17.8	20.2	67.44	6068	9.49	17.7	19.7	73.94	9.34	17.6	19.2	80.44	9.20	17.5	18.6
" ¾	66.26	6385	9.82	17.8	20.8	72.76	6751	9.64	17.8	20.2	79.26	9.47	17.7	19.7	85.76	9.34	17.6	19.1
" ⅞	71.42	7043	9.94	17.9	21.3	77.92	7409	9.76	17.9	20.8	84.42	9.60	17.8	20.2	90.92	9.46	17.7	19.6
8x8x½	63.50	5818	9.58	16.8	20.5	70.00	6184	9.40	16.8	20.0	76.50	9.26	16.8	19.4	83.00	9.13	16.8	18.8
" ⅝	70.94	6737	9.75	16.8	21.0	77.44	7103	9.58	16.8	20.4	83.94	9.44	16.8	19.9	90.44	9.32	16.8	19.3
" ¾	78.26	7617	9.88	16.8	21.6	84.76	7983	9.71	16.7	20.9	91.26	9.56	16.7	20.4	97.76	9.45	16.7	19.8
" ⅞	85.42	8471	9.96	16.7	22.0	91.92	8837	9.81	16.6	21.4	98.42	9.67	16.6	20.9	104.92	9.56	16.6	20.3
" 1	92.50	9289	10.02	16.6	22.3	99.00	9655	9.88	16.6	21.9	105.50	9.76	16.6	21.4	112.00	9.64	16.6	20.8
	28″ x ¾″ Web Plates.					28″ x ⅞″ Web Plates.					28″ x 1″ Web Plates.				28″ x 1⅛″ Web Plates.			
4x4x⅜	53.44	4728	9.41	18.8	18.6	60.44	5185	9.27	18.8	18.4	67.44	9.15	18.7	17.8	74.44	9.05	18.6	17.4
" ½	57.00	5329	9.67	19.1	19.3	64.00	5786	9.51	19.0	18.9	71.00	9.38	19.0	18.3	78.00	9.27	18.9	18.0
" ⅝	60.44	5898	9.88	19.4	19.9	67.44	6355	9.71	19.3	19.5	74.44	9.57	19.2	18.9	81.44	9.45	19.1	18.5
6x6x½	65.00	6458	9.97	19.0	20.4	72.00	6915	9.81	18.9	19.9	79.00	9.66	18.9	19.5	86.00	9.55	18.8	19.0
" ⅝	70.44	7299	10.17	19.1	20.9	77.44	7756	10.01	19.0	20.4	84.44	9.87	19.0	20.0	91.44	9.74	18.9	19.5
" ¾	75.76	8106	10.35	19.2	21.5	82.76	8563	10.21	19.1	21.0	89.76	10.03	19.1	20.5	96.76	9.90	19.0	20.0
" ⅞	80.92	8885	10.47	19.3	22.0	87.92	9342	10.31	19.2	21.5	94.92	10.16	19.2	21.0	101.92	10.03	19.1	20.5
8x8x½	73.00	7447	10.10	18.3	21.2	80.00	7904	9.94	18.3	20.7	87.00	9.81	18.4	20.2	94.00	9.69	18.4	19.7
" ⅝	80.44	8536	10.30	18.3	21.8	87.44	8993	10.14	18.3	21.2	94.44	10.00	18.4	20.7	101.44	9.90	18.4	20.3
" ¾	87.76	9579	10.45	18.3	22.4	94.76	10036	10.30	18.3	21.7	101.76	10.15	18.4	21.2	108.76	10.03	18.4	20.9
" ⅞	94.92	10594	10.56	18.3	22.8	101.92	11051	10.42	18.3	22.2	108.92	10.27	18.4	21.7	115.92	10.06	18.4	21.3
" 1	102.00	11568	10.65	18.3	23.3	109.00	12025	10.50	18.4	22.8	116.00	10.37	18.4	22.3	123.00	10.25	18.4	21.8
	30″ x ¾″ Web Plates.					30″ x ⅞″ Web Plates.					30″ x 1″ Web Plates.				30″ x 1⅛″ Web Plates.			
4x4x⅜	56.44	5670	10.02	20.1	19.9	63.94	6233	9.88	20.0	19.5	71.44	9.76	20.0	19.0	78.94	9.56	19.9	18.6
" ½	60.00	6367	10.30	20.5	20.6	67.50	6930	10.12	20.4	20.0	75.00	10.00	20.3	19.6	82.50	9.89	20.2	19.2
" ⅝	63.44	7027	10.51	20.8	21.2	70.94	7590	10.35	20.7	20.5	78.44	10.20	20.5	20.2	85.94	10.06	20.4	19.7
6x6x½	68.00	7690	10.64	20.5	21.7	75.50	8253	10.46	20.4	21.2	83.00	10.30	20.3	20.8	90.50	10.18	20.2	20.3
" ⅝	73.44	8670	10.86	20.7	22.2	80.94	9233	10.68	20.6	21.8	88.44	10.51	20.5	21.4	95.94	10.40	20.4	20.8
" ¾	78.76	9613	11.05	20.9	22.8	86.26	10176	10.86	20.7	22.3	93.76	10.70	20.6	21.9	101.26	10.56	20.5	21.4
" ⅞	83.92	10522	11.20	21.0	23.4	91.42	11085	11.02	20.9	22.9	98.92	10.85	20.8	22.5	106.42	10.71	20.7	21.9
8x8x½	76.00	8857	10.78	19.9	22.5	83.50	9420	10.62	20.0	22.0	91.00	10.46	19.8	21.5	98.50	10.35	19.9	21.1
" ⅝	83.44	10129	11.02	19.9	23.0	90.94	10692	10.85	20.1	22.5	98.44	10.70	19.8	22.0	105.94	10.56	20.0	21.8
" ¾	90.76	11352	11.20	19.9	23.6	98.26	11915	11.02	20.2	23.1	105.76	10.85	19.8	22.6	113.26	10.73	20.1	22.4
" ⅞	97.92	12541	11.32	20.0	24.1	105.42	13104	11.15	20.2	23.6	112.92	11.00	19.9	23.1	120.42	10.90	20.1	22.9
" 1	105.00	13685	11.42	20.0	24.7	112.50	14248	11.25	20.2	24.2	120.00	11.11	19.9	23.7	127.50	10.98	20.1	23.4

TABLE 71.—*Continued.*

PROPERTIES OF FOUR ANGLES AND TWO PLATES, LACED.

Properties of Four Angles and Two Plates, Laced. Angles Turned Out and Angles Turned In.



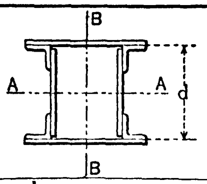
b = Width, Back to Back of Angles, for Equal Moments of Inertia about Axes A–A and B–B with Angles Turned Out.
c = Same as b, for Angles Turned In.
d = Depth of Web Plates + ½″.

Series 1, 2, 3 and 4. Size of Angles.	Series 1. Total Area.	Series 1. Moment of Inertia.	Series 1. Radius of Gyration.	Series 1. b. to b. Angles.		Series 2. Total Area.	Series 2. Moment of Inertia.	Series 2. Radius of Gyration.	Series 2. b. to b. Angles.		Series 3. Total Area.	Series 3. Radius of Gyration.	Series 3. b. to b. Angles.		Series 4. Total Area.	Series 4. Radius of Gyration.	Series 4. b. to b. Angles.	
	A	I	r	b	c	A	I	r	b	c	A	r	b	c	A	r	b	c
In.	In.²	In.⁴	In.	In.	In.	In.²	In.⁴	In.	In.	In.	In.²	In.	In.	In.	In.²	In.	In.	In.
	32″x¾″ Web Plates.					32″x⅞″ Web Plates.					32″x1″ Web Plates.				32″x1⅛″ Web Plates.			
4x4x⅜	59.44	6725	10.65	21.4	21.1	67.44	7408	10.47	21.3	20.7	75.44	10.35	21.2	20.2	83.44	10.25	21.1	19.8
" ½	63.00	7525	10.94	21.8	21.8	71.00	8208	10.75	21.7	21.3	79.00	10.60	21.6	20.8	87.00	10.50	21.4	20.4
" ⅝	66.44	8284	11.16	22.1	22.4	74.44	8967	10.97	22.0	21.9	82.44	10.82	21.8	21.4	90.44	10.70	21.7	20.9
6x6x½	71.00	9058	11.30	21.8	23.0	79.00	9741	11.11	21.7	22.5	87.00	10.95	21.6	21.9	95.00	10.80	21.5	21.4
" ⅝	76.44	10189	11.55	22.0	23.6	84.44	10872	11.35	21.9	23.1	92.44	11.18	21.8	22.5	100.44	11.04	21.7	22.0
" ¾	81.76	11277	11.75	22.2	24.2	89.76	11960	11.55	22.1	23.6	97.76	11.37	21.9	23.1	105.76	11.23	21.8	22.5
" ⅞	86.92	12328	11.90	22.4	24.8	94.92	13011	11.72	22.3	24.2	102.92	11.54	22.1	23.7	110.92	11.38	22.0	23.1
8x8x½	79.00	10419	11.50	21.3	23.9	87.00	11102	11.30	21.3	23.3	95.00	11.14	21.2	22.8	103.00	11.00	21.2	22.2
" ⅝	86.44	11890	11.74	21.4	24.6	94.44	12573	11.55	21.4	24.0	102.00	11.40	21.3	23.3	110.44	11.25	21.3	22.9
" ¾	93.76	13305	11.92	21.6	25.3	101.76	13988	11.71	21.5	24.7	109.76	11.55	21.4	24.1	117.76	11.42	21.3	23.5
" ⅞	100.92	14683	12.06	21.6	25.8	108.92	15366	11.89	21.5	25.2	116.92	11.72	21.4	24.6	124.92	11.57	21.3	24.0
" 1	108.00	16011	12.18	21.6	26.2	116.00	16694	12.00	21.5	25.6	124.00	11.85	21.4	25.1	132.00	11.70	21.3	24.5
	34″x¾″ Web Plates.					34″x⅞″ Web Plates.					34″x1″ Web Plates.				34″x1⅛″ Web Plates.			
4x4x⅜	62.44	7899	11.25	22.6	22.2	70.94	8718	11.08	22.5	21.8	79.44	10.95	22.4	21.4	87.94	10.85	22.3	21.0
" ½	66.00	8809	11.55	23.0	22.9	74.50	9628	11.37	22.9	22.5	83.00	11.21	22.8	22.0	91.50	11.10	22.7	21.6
" ⅝	69.44	9673	11.80	23.4	23.7	77.94	10492	11.60	23.3	23.2	86.44	11.45	23.1	22.6	94.94	11.30	23.0	22.1
6x6x½	74.00	10568	11.95	23.2	24.3	82.50	11387	11.75	23.0	23.8	91.00	11.58	22.9	23.3	99.50	11.45	22.7	22.8
" ⅝	79.44	11860	12.23	23.4	24.9	87.94	12679	12.02	23.2	24.3	96.44	11.84	23.1	23.8	104.94	11.70	22.9	23.3
" ¾	84.76	13105	12.45	23.7	25.6	93.26	13924	12.23	23.5	25.0	101.76	12.03	23.4	24.5	110.26	11.89	23.2	23.9
" ⅞	89.92	14307	12.63	23.9	26.2	98.42	15126	12.37	23.7	25.7	106.92	12.20	23.6	25.2	115.42	12.05	23.4	24.4
8x8x½	82.00	12138	12.16	22.8	25.2	90.50	12957	11.97	22.7	24.6	99.00	11.80	22.6	24.1	107.50	11.65	22.5	23.5
" ⅝	89.44	13823	12.44	22.9	25.9	97.94	14642	12.24	22.9	25.4	106.44	12.06	22.8	24.8	114.94	11.90	22.7	24.2
" ¾	96.76	15447	12.65	23.1	26.7	105.26	16266	12.44	23.0	26.1	113.76	12.25	22.9	25.5	122.26	12.10	22.8	24.9
" ⅞	103.92	17027	12.81	23.1	27.2	112.42	17846	12.60	23.0	26.6	120.92	12.44	23.0	26.0	129.42	12.28	22.9	25.4
" 1	111.00	18554	12.97	23.2	27.7	119.50	19373	12.75	23.1	27.1	128.00	12.55	23.1	26.5	136.50	12.40	23.0	25.9
	36″x¾″ Web Plates.					36″x⅞″ Web Plates.					36″x1″ Web Plates.				36″x1⅛″ Web Plates.			
4x4x⅜	65.44	9199	11.85	23.9	23.4	74.44	10171	11.70	23.9	23.0	83.44	11.55	23.9	22.7	92.44	11.45	23.5	22.3
" ½	69.00	10225	12.18	24.3	24.1	78.00	11197	11.97	24.2	23.7	87.00	11.84	24.2	23.3	96.00	11.70	23.8	22.8
" ⅝	72.44	11201	12.45	24.7	24.9	81.44	12173	12.[illegible]3	24.5	24.4	90.44	12.06	24.4	23.8	99.44	11.91	24.2	23.3
6x6x½	77.00	12227	12.60	24.6	25.5	86.00	13199	12.40	24.4	25.0	95.00	12.22	24.3	24.4	104.00	12.06	24.1	23.9
" ⅝	82.44	13690	12.85	24.8	26.2	91.44	14662	12.66	24.8	25.8	100.44	12.48	24.7	25.3	109.44	12.30	24.7	24.9
" ¾	87.76	15102	13.12	25.1	26.8	96.76	16074	12.90	25.1	26.5	105.76	12.70	24.8	25.7	114.76	12.54	25.2	25.9
" ⅞	92.92	16466	13.32	25.3	27.5	101.92	17438	13.08	25.5	26.9	110.92	12.90	25.0	26.3	119.92	12.71	25.8	26.9
8x8x½	85.00	14022	12.85	24.3	26.5	94.00	14994	12.64	24.2	25.9	103.00	12.45	24.0	25.3	112.00	12.30	23.9	24.7
" ⅝	92.44	15935	13.14	24.5	27.3	101.44	16907	12.92	24.4	26.6	110.44	12.74	24.2	26.1	119.44	12.57	24.1	25.4
" ¾	99.76	17782	13.36	24.7	28.1	108.76	18754	13.14	24.6	27.4	117.76	12.95	24.4	26.8	126.76	12.78	24.3	26.1
" ⅞	106.92	19580	13.55	24.7	28.6	115.92	20552	13.32	24.6	28.0	124.92	13.09	24.5	27.3	133.92	12.96	24.4	26.7
" 1	114.00	21318	13.69	24.8	29.1	123.00	22290	13.45	24.7	28.5	132.00	13.25	24.6	27.8	141.00	13.12	24.5	27.2

TABLE 72.

Properties of Four Angles and Four Plates.

Properties of Four Angles and Four Plates.



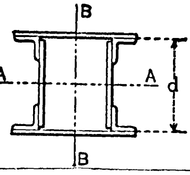
Edges of Angles Flush with Edges of Cover Plates.
d = Depth of Web Plates Plus ½″.

Series 1, 2 and 3.		Series 1.					Series 2.					Series 3.				
			Axis A–A.		Axis B–B.			Axis A–A.		Axis B–B.			Axis A–A.		Axis B–B	
Size of Angles.	Cover Plates.	Total Area.	Moment of Inertia.	Radius of Gyration.	Moment of Inertia.	Radius of Gyration.	Total Area.	Moment of Inertia.	Radius of Gyration.	Moment of Inertia.	Radius of Gyration.	Total Area.	Moment of Inertia.	Radius of Gyration	Moment of Inertia.	Radius of Gyration.
		A	I_A	r_A	I_B	r_B	A	I_A	r_A	I_B	r_B	A	I_A	r_A	I_B	r_B
In.	In.	In.²	In.⁴	In.	In.⁴	In.	In.²	In.⁴	In.	In.⁴	In.	In.²	In.⁴	In.	In.⁴	In.
		12″ × ⅜″ Web Plates.					12″ × ½″ Web Plates.					12″ × ⅝″ Web Plates.				
3x3x¼	14x⅜	25.26	717	5.32	442	4.19	28.26	753	5.16	481	4.13	31.26	789	5.02	516	4.06
"	" ½	28.76	874	5.51	499	4.17	31.76	910	5.35	538	4.12	34.76	946	5.22	573	4.06
"	" ⅝	32.26	1037	5.67	557	4.15	35.26	1073	5.52	595	4.11	38.26	1109	5.39	630	4.06
3x3x⅜	14x⅜	27.94	793	5.33	511	4.28	30.94	829	5.18	550	4.22	33.94	865	5.05	585	4.15
"	" ½	31.44	950	5.50	568	4.26	34.44	986	5.35	607	4.19	37.44	1022	5.23	642	4.14
"	" ⅝	34.94	1113	5.65	626	4.23	37.94	1149	5.53	664	4.18	40.94	1185	5.38	699	4.13
3½x3½x⅜	16x⅜	30.92	890	5.36	737	4.88	33.92	926	5.22	786	4.81	36.92	962	5.10	833	4.75
"	" ½	34.92	1069	5.53	822	4.85	37.92	1105	5.40	871	4.79	40.92	1141	5.28	918	4.73
"	" ⅝	38.92	1254	5.68	907	4.83	41.92	1290	5.55	956	4.78	44.92	1326	5.43	1003	4.72
3½x3½x½	16x⅜	34.00	971	5.34	840	4.97	37.00	1007	5.22	890	4.91	40.00	1043	5.11	936	4.84
"	" ½	38.00	1150	5.52	926	4.94	41.00	1186	5.38	975	4.88	44.00	1222	5.27	1022	4.82
"	" ⅝	42.00	1335	5.64	1011	4.92	45.00	1371	5.52	1060	4.86	48.00	1407	5.41	1107	4.81
		14″ × ⅜″ Web Plates.					14″ × ½″ Web Plates.					14″ × ⅝″ Web Plates.				
3½x3½x⅜	18x⅜	33.92	1317	6.24	1093	5.68	37.42	1374	6.06	1183	5.62	40.92	1431	5.91	1268	5.57
"	" ½	38.42	1583	6.42	1215	5.63	41.92	1640	6.26	1304	5.58	45.42	1697	6.12	1390	5.54
"	" ⅝	42.92	1857	6.58	1336	5.58	46.42	1914	6.42	1426	5.54	49.92	1971	6.28	1511	5.51
3½x3½x½	18x⅜	37.00	1432	6.22	1235	5.78	40.50	1489	6.07	1325	5.72	44.00	1546	5.93	1410	5.66
"	" ½	41.50	1698	6.40	1357	5.72	45.00	1755	6.30	1446	5.67	48.50	1812	6.12	1532	5.62
"	" ⅝	46.00	1972	6.55	1478	5.67	49.50	2029	6.41	1568	5.63	53.00	2086	6.28	1653	5.60
4x4x⅜	18x⅜	35.44	1363	6.20	1057	5.47	38.94	1415	6.03	1130	5.39	42.44	1473	5.89	1198	5.33
"	" ½	39.94	1629	6.39	1178	5.44	43.44	1686	6.23	1251	5.37	46.94	1743	6.10	1320	5.30
"	" ⅝	44.44	1903	6.55	1300	5.41	47.94	1960	6.42	1373	5.35	51.44	2017	6.26	1441	5.29
4x4x½	18x⅜	39.00	1494	6.19	1203	5.56	42.50	1551	6.04	1276	5.48	46.00	1608	5.91	1345	5.41
"	" ½	43.50	1760	6.36	1325	5.52	47.00	1817	6.22	1397	5.45	50.50	1874	6.09	1466	5.39
"	" ⅝	48.00	2034	6.51	1446	5.49	51.50	2091	6.3°	1519	5.43	55.00	2148	6.25	1588	5.38
		16″ × ⅜″ Web Plates.					16″ × ½″ Web Plates.					16″ × ⅝″ Web Plates.				
3½x3½x⅜	20x½	41.92	2234	7.30	1716	6.40	45.92	2319	7.11	1863	6.37	49.92	2405	6.94	2004	6.34
"	" ⅝	46.92	2622	7.48	1883	6.34	50.92	2707	7.29	2030	6.32	54.92	2793	7.13	2171	6.29
"	" ¾	51.92	3022	7.63	2049	6.28	55.92	3107	7.46	2196	6.27	59.92	3193	7.30	2337	6.25
3½x3½x½	20x½	45.00	2389	7.29	1903	6.50	49.00	2474	7.11	2050	6.47	53.00	2560	6.95	2191	6.43
"	" ⅝	50.00	2777	7.45	2069	6.43	54.00	2862	7.28	2217	6.41	58.00	2948	7.14	2357	6.38
"	" ¾	55.00	3177	7.56	2236	6.45	59.00	3262	7.44	2383	6.35	63.00	3348	7.30	2524	6.33
4x4x⅜	20x½	43.44	2298	7.28	1674	6.21	47.44	2383	7.09	1797	6.16	51.44	2469	6.93	1915	6.10
"	" ⅝	48.44	2686	7.44	1840	6.16	52.44	2771	7.27	1964	6.12	56.44	2857	7.12	2082	6.07
"	" ¾	53.44	3086	7.60	2007	6.13	57.44	3171	7.43	2130	6.09	61.44	3257	7.28	2249	6.05
4x4x½	20x½	47.00	2474	7.26	1869	6.31	51.00	2559	7.09	1992	6.25	55.00	2645	6.94	2110	6.20
"	" ⅝	52.00	2862	7.42	2035	6.26	56.00	2947	7.26	2158	6.21	60.00	3033	7.11	2277	6.16
"	" ¾	57.00	3262	7.55	2202	6.22	61.00	3347	7.41	2325	6.19	65.00	3433	7.27	2444	6.13

TABLE 72.—*Continued.*

PROPERTIES OF FOUR ANGLES AND FOUR PLATES.

Properties of Four Angles and Four Plates.



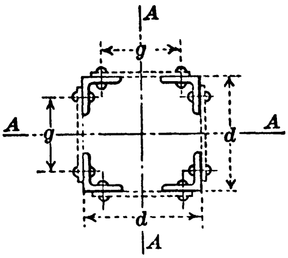
Edges of Angles Flush with Edges of Cover Plates.
d = Depth of Web Plates Plus ½″.

Series 1, 2 and 3.		Series 1.					Series 2.					Series 3.				
			Axis A–A.		Axis B–B.			Axis A–A.		Axis B–B.			Axis A–A.		Axis B–B.	
Size of Angles.	Cover Plates.	Total Area.	Moment of Inertia.	Radius of Gyration.	Moment of Inertia.	Radius of Gyration.	Total Area.	Moment of Inertia.	Radius of Gyration.	Moment of Inertia.	Radius of Gyration.	Total Area.	Moment of Inertia.	Radius of Gyration.	Moment of Inertia.	Radius of Gyration.
		A	I_A	r_A	I_B	r_B	A	I_A	r_A	I_B	r_B	A	I_A	r_A	I_B	r_B
In.	In	In.²	In.⁴	In.	In.⁴	In.	In.²	In.⁴	In.	In.⁴	In.	In.²	In.⁴	In.	In.⁴	In.
		18″ × ½″ Web Plates.					18″ × ⅝″ Web Plates.					18″ × ¾″ Web Plates.				
3½x3½x⅜	22x½	49.92	3158	7.97	2564	7.17	54.42	3279	7.76	2780	7.16	58.92	3401	7.60	2989	7.13
"	" ⅝	55.42	3686	8.15	2786	7.10	59.92	3807	7.98	3002	7.11	64.42	3929	7.81	3211	7.06
"	" ¾	60.92	4229	8.34	3008	7.03	65.42	4351	8.16	3224	7.02	69.92	4472	8.00	3432	7.01
3½x3½x½	22x½	53.00	3360	7.96	2802	7.27	57.50	3481	7.79	3018	7.25	62.00	3603	7.63	3226	7.22
"	" ⅝	58.50	3888	8.16	3023	7.20	63.00	4009	7.98	3239	7.17	67.50	4131	7.82	3448	7.15
"	" ¾	64.00	4431	8.32	3245	7.13	68.50	4553	8.15	3461	7.11	73.00	4674	8.00	3670	7.09
4x4x⅜	22x½	51.44	3243	7.94	2484	6.95	55.94	3364	7.76	2669	6.91	60.44	3486	7.59	2849	6.87
"	" ⅝	56.94	3771	8.14	2705	6.89	61.44	3892	7.96	2891	6.86	65.94	4014	7.80	3071	6.82
"	" ¾	62.44	4314	8.32	2927	6.85	66.94	4436	8.14	3113	6.81	71.44	4557	8.00	3293	6.79
4x4x½	22x½	55.00	3472	7.95	2734	7.06	59.50	3593	7.77	2919	7.01	64.00	3715	7.62	3099	6.96
"	" ⅝	60.50	4000	8.13	2956	7.00	65.00	4121	7.96	3141	6.95	69.50	4243	7.80	3321	6.92
"	" ¾	66.00	4543	8.30	3178	6.94	70.50	4665	8.14	3363	6.91	75.00	4786	8.00	3543	6.88
		20″ × ½″ Web Plates.					20″ × ⅝″ Web Plates.					20″ × ¾″ Web Plates.				
3½x3½x½	24x½	57.00	4426	8.83	3717	8.08	62.00	4593	8.61	4031	8.07	67.00	4759	8.45	4337	8.04
"	" ⅝	63.00	5127	9.02	4005	7.98	68.00	5293	8.83	4319	7.98	73.00	5460	8.65	4625	7.96
"	" ¾	69.00	5844	9.22	4293	7.88	74.00	6011	9.01	4607	7.89	79.00	6178	8.85	4913	7.89
3½x3½x⅝	24x½	59.92	4664	8.82	3999	8.18	64.92	4831	8.62	4313	8.15	69.92	4997	8.46	4619	8.12
"	" ⅝	65.92	5365	9.02	4287	8.06	70.92	5531	8.84	4601	8.06	75.92	5698	8.67	4907	8.04
"	" ¾	71.92	6082	9.22	4575	7.98	76.92	6249	9.02	4889	7.97	81.92	6416	8.86	5195	7.96
4x4x½	24x½	59.00	4571	8.80	3640	7.86	64.00	4737	8.60	3916	7.84	69.00	4903	8.44	4184	7.79
"	" ⅝	65.00	5271	9.01	3928	7.77	70.00	5437	8.82	4204	7.78	75.00	5604	8.65	4472	7.73
"	" ¾	71.00	5988	9.18	4216	7.71	76.00	6155	9.01	4492	7.70	81.00	6322	8.84	4760	7.67
4x4x⅝	24x½	62.44	4841	8.80	3952	7.96	67.44	5008	8.62	4228	7.92	72.44	5174	8.45	4496	7.88
"	" ⅝	68.44	5542	9.00	4240	7.87	73.44	5708	8.82	4516	7.84	78.44	5875	8.66	4784	7.80
"	" ¾	74.44	6259	9.17	4528	7.80	79.44	6426	9.00	4804	7.78	84.44	6593	8.85	5072	7.76
		22″ × ½″ Web Plates.					22″ × ⅝″ Web Plates.					22″ × ¾″ Web Plates.				
3½x3½x½	28x⅝	70.00	6933	9.96	6351	9.53	75.50	7155	9.74	6894	9.56	81.00	7377	9.55	7422	9.58
"	" ¾	77.00	7930	10.15	6808	9.40	82.50	8152	9.94	7351	9.44	88.00	8373	9.76	7879	9.47
"	" ⅞	84.00	8949	10.32	7265	9.31	89.50	9171	10.13	7809	9.35	95.00	9393	9.95	8337	9.37
3½x3½x⅝	28x⅝	72.92	7226	9.96	6758	9.63	78.42	7448	9.75	7302	9.65	83.92	7670	9.56	7830	9.66
"	" ¾	79.92	8223	10.15	7216	9.51	85 42	8445	9.95	7759	9.54	90.92	8666	9.76	8287	9.56
"	" ⅞	86.92	9242	10.31	7673	9.40	92.42	9464	10.13	8217	9.43	97.92	9686	9.95	8745	9.45
4x4x½	28x⅝	72.00	7112	9.95	6276	9.34	77.50	7334	9.74	6764	9.35	83.00	7556	9.55	7242	9.35
"	" ¾	79.00	8109	10.13	6733	9.24	84.50	8331	9.94	7222	9.25	90.00	8552	9.75	7699	9.25
"	" ⅞	86.00	9128	10.30	7191	9.15	91.50	9350	10.11	7679	9.16	97.00	9572	10.04	8157	9.17
4x4x⅝	28x⅝	75.44	7448	9.94	6731	9.45	80 94	7670	9.74	7219	9.45	86.44	7892	9.56	7697	9.45
"	" ¾	82.44	8445	10.12	7188	9.34	87 94	8667	9.94	7677	9.35	93.44	8888	9.76	8154	9.35
"	" ⅞	89.44	9464	10.28	7646	9.26	94.94	9686	10.11	8134	9.26	100.44	9908	9.96	8612	9.26

TABLE 73.

Properties of Four Angles Laced and Eight Angles Battened.

Four Angles.



Laced (Box Column).

Size of Angles.	Area of Four Angles.	Axis A–A. Moment of Inertia.	Radius of Gyration.	Moment of Inertia.	Radius of Gyration.	Moment of Inertia.	Radius of Gyration.
		I_A	r_A	I_A	r_A	I_A	r_A
In.	In.^{2}	In.^{4}	In.	In.^{4}	In.	In.^{4}	In.
		Value of d in Inches.					
		8½		10½		12½	
3x3x¼	5.76	72	3.53	117	4.50	174	5.49
" ⅜	8.44	102	3.48	167	4.45	249	5.44
" ½	11.00	130	3.44	214	4.41	320	5.39
		10½		12½		14½	
3½x3½x⅜	9.92	190	4.38	284	5.35	398	6.33
" ½	13.00	243	4.32	365	5.30	513	6.28
" ⅝	15.92	291	4.28	440	5.26	620	6.24
		12½		14½		16½	
4x4x⅜	11.44	316	5.26	444	6.23	596	7.22
" ½	15.00	408	5.22	575	6.19	772	7.17
" ⅝	18.44	491	5.16	695	6.14	935	7.12
		16½		18½		20½	
6x6x⅜	17.44	824	6.87	1072	7.84	1354	8.81
" ½	23.00	1072	6.82	1398	7.79	1769	8.76
" ⅝	28.44	1306	6.77	1705	7.74	2161	8.72
" ¾	33.76	1526	6.72	1996	7.68	2535	8.66

The table given above is intended to serve only as a guide in the choice of sections and not as a complete table. The properties of other sections may be found as follows:

Example: Required the properties of a square box column consisting of 4 ∠ 4″x4″x½″, laced, 13½ in. back to back.

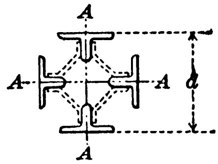
Solution: Table 32 evidently applies to angles with legs turned in, as well as angles with legs turned out.

Area, from Table 32 = 15.00 in.^{2}

$I_A = I_x$, from Table 32 = 467 in.^{4}

$r_A = \sqrt{I_A \div A} = \sqrt{467 \div 15.00} = 5.58$ in.

Eight Angles.



Battened (Gray Column).

Size of Angles.	Area of Eight Angles.	Axis A–A. Moment of Inertia.	Radius of Gyration.	Moment of Inertia.	Radius of Gyration.	Moment of Inertia.	Radius of Gyration.
		I_A	r_A	I_A	r_A	I_A	r_A
In.	In.^{2}	In.^{4}	In.	In.^{4}	In.	In.^{4}	In.
		Value of d in Inches.					
		12½		14½		16½	
3x3x¼	11.52	183	3.97	251	4.67	330	5.35
" ⅜	16.88	263	3.95	362	4.63	478	5.32
" ½	22.00	338	3.92	466	4.60	616	5.29
		12½		14½		16½	
3½x3½x⅜	19.84	306	3.93	419	4.59	553	5.28
" ½	26.00	394	3.89	542	4.57	716	5.25
" ⅝	31.84	476	3.87	656	4.54	868	5.22
		14½		16½		18½	
4x4x⅜	22.88	477	4.56	628	5.24	802	5.92
" ½	30.00	618	4.54	815	5.21	1042	5.89
" ⅝	36.88	750	4.51	990	5.18	1267	5.86
		18½		20½		22½	
6x6x⅜	34.88	1180	5.82	1463	6.48	1781	7.14
" ½	46.00	1542	5.79	1914	6.45	2331	7.12
" ⅝	56.88	1887	5.76	2343	6.42	2856	7.08
" ¾	67.52	2216	5.73	2755	6.39	3360	7.05

The table given above is intended to serve only as a guide in the choice of sections and not as a complete table. The properties of other sections may be found as follows:

Example: Required the properties of a column consisting of 8 ∠ 4″x4″x½″, battened, 15½ in. back to back.

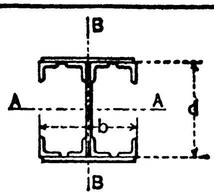
Solution: From Tables 32 and 35 the moment of inertia about axis A–A equals 645 + 43 = 688 in.^{4} and the area equals 2 × 15.00 = 30.00 sq. in.

The radius of gyration equals

$r = \sqrt{I \div A} = \sqrt{688 \div 30.00} = 4.79$ in.

TABLE 74.

Properties of Eight Angles and Three Plates.

Properties of Eight Angles and Three Plates.		d = Width of Web Plate Plus One-half Inch. b = Width of Flange Plates Plus One-half Inch. Large Sections may be Laced on Open Sides.

Size of Web Plate.	Size of Flange Plates.	Size of Inside Angles.	Size of Outside Angles.	Total Area	Axis A–A. Moment of Inertia.	Axis A–A. Radius of Gyration.	Axis B–B. Moment of Inertia.	Axis B–B. Radius of Gyration.
				A	I_A	r_A	I_B	r_B
In.	In.	In.	In.	In.²	In.⁴	In.	In.⁴	In.
18x½	18x½	3½x3½x⅜	3½x3½x⅜	46.84	3238	8.31	1198	5.06
" ⅝	" ⅝	" ½	" ½	59.75	4135	8.32	1534	5.07
" ¾	" ¾	" ⅝	" ⅝	72.34	5016	8 32	1856	5.06
20x½	20x½	4x4x½	4x4x½	60.00	5051	9.17	1976	5.74
" ⅝	" ⅝	" ⅝	" ⅝	74.38	6261	9.17	2431	5.71
" ¾	" ¾	" ¾	" ¾	88.52	7459	9.18	2875	5.70
22x⅝	22x⅝	4x4x½	4x4x½	71.24	7319	10.13	2708	6.16
" ¾	" ¾	" ⅝	" ⅝	86.37	8885	10.14	3285	6.16
" ⅞	" ⅞	" ¾	" ¾	101.26	10434	10.15	3845	6.16
24x⅝	24x⅝	4x4x½	4x4x½	75.00	9175	11.05	3356	6.69
" ¾	" ¾	" ⅝	" ⅝	90.88	11139	11.06	4070	6.69
" ⅞	" ⅞	" ¾	" ¾	106.52	13083	11.08	4767	6.68
26x¾	26x¾	6x6x¾	6x6x¾	126.02	17447	11.77	7021	7.46
" ⅞	" ⅞	" ⅞	" ⅞	146.09	20234	11.77	8102	7.44
" 1	" 1	" 1	" 1	166.00	23001	11.77	9168	7.43
28x¾	28x¾	6x6x¾	6x6x¾	130.52	21081	12.71	8376	8.01
" ⅞	" ⅞	" ⅞	" ⅞	151.34	24456	12.71	9672	7.99
" 1	" 1	" 1	" 1	172.00	27809	12.71	10943	7.98
30x⅞	30x⅞	6x6x¾	6x6x¾	146.27	27369	13.67	10456	8.45
" 1	" 1	" ⅞	" ⅞	167.84	31433	13.68	11988	8.45
" 1⅛	" 1⅛	" 1	" 1	189.25	35477	13.69	13496	8.45

The above table is intended to serve only as a guide in the choice of sections and not as a complete table. The properties of other sections may be found as follows:

Example: Required the properties of a section composed of a 20″ × ⅝″ web plate, two 24″ × ¾″ flange plates, four 4″ × 4″ × ½″ inside angles and, four 6″ × 4″ × ¾″ outside angles fastened by 4″ legs, $d = 20\frac{1}{2}''$, $b = 24\frac{1}{2}''$.

Solution:

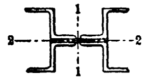
Item.		Area. Table No.	Area. A	Moment of Inertia. Axis A–A. Table No.	Moment of Inertia. Axis A–A. I_A	Moment of Inertia. Axis B–B. Table No.	Moment of Inertia. Axis B–B. I_B	Radius of Gyration. Axis A–A. $r_A = \sqrt{I_A \div A}$	Radius of Gyration. Axis B–B. $r_B = \sqrt{I_B \div A}$
In.			In.²		In ⁴		In.⁴	In.	In
1-Wb. Pl.	20x⅝	1	12 50	3	417	4	0	$\sqrt{\frac{7506}{91.26}}$	$\sqrt{\frac{5205}{91.26}}$
2-Fl. Pls.	24x¾	1	36.00	5	3972	3	1728		
4-Ins. ∠	4x4x½	32	15.00	32	1222	35	56		
4-Outs. ∠	6x4x¾	34	27.76	34	1895	33	3421		
Total . .	....	A =	91.26	I_A =	7506	I_B =	5205	r_A = 9.07	r_B = 7.55

TABLE 75

Elements of Z-Bar Columns

American Bridge Company Standards

Dimensions in Inches



Rivets ¾" Diam.

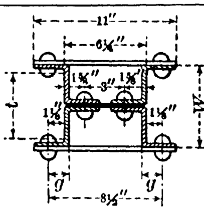
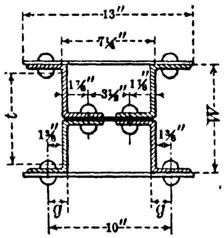
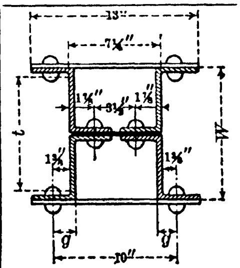
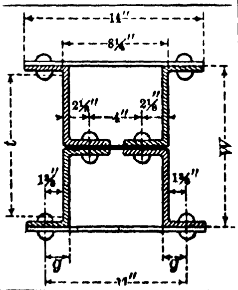
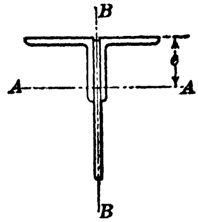
Size of Column	Size of Web Pl.	Size of Z-Bars: Thickness	Size of Z-Bars: Size of Flanges	Width W	Gage g	Tang't t	Standard Dimensions	Axis 1-1: Moment of Inertia	Axis 1-1: Radius of Gyration	Axis 2-2: Moment of Inertia	Axis 2-2: Radius of Gyration	Weight per Foot	Area
In.		In.	Ins.	In.	In.	In.						Lbs.	Sq. In.
6	6" Web Same Thickness as Z-Bar	1/4	2 5/8 × 3 × 2 5/8	6 1/4	1 3/8	5 3/8		84.7	3.0	31.7	1.8	31.5	9.26
		5/16	2 11/16 × 3 1/16 × 2 11/16	6 7/16	1 7/16	5 7/16		105.1	3.0	41.8	1.9	39.6	11.64
		3/8	2 3/4 × 3 1/8 × 2 3/4	6 5/8	1 1/2	5 1/2		125.1	2.9	53.4	1.9	47.6	14.01
		7/16	2 11/16 × 3 × 2 11/16	6 7/16	1 9/16	4 13/16		134.6	2.9	55.2	1.8	53.5	15.63
		1/2	2 3/4 × 3 1/16 × 2 3/4	6 5/8	1 5/8	4 7/8		153.1	2.9	67.1	1.9	61.2	18.00
8	7" Web Same Thickness as Z-Bar	1/4	2 7/8 × 4 × 2 7/8	8 1/4	1 5/8	7 1/8		134.7	3.4	65.7	2.4	37.5	11.03
		5/16	2 15/16 × 4 1/16 × 2 15/16	8 7/16	1 11/16	7 3/16		166.9	3.4	85.8	2.4	47.0	13.83
		3/8	3 × 4 1/8 × 3	8 5/8	1 3/4	7 1/4		199.4	3.4	107.8	2.5	56.5	16.71
		7/16	3 × 4 × 3	8 7/16	1 13/16	6 13/16		220.6	3.4	115.6	2.4	64.3	18.90
		1/2	3 1/16 × 4 1/16 × 3 1/16	8 5/8	1 7/8	6 7/8		250.8	3.4	138.6	2.5	73.9	21.74
		9/16	3 1/8 × 4 1/8 × 3 1/8	8 13/16	1 15/16	6 15/16		280.4	3.3	163.0	2.5	83.6	24.58
		5/8	3 1/16 × 4 × 3 1/16	8 5/8	2	6 1/2		296.3	3.3	167.3	2.5	90.1	26.58
		11/16	3 1/8 × 4 1/16 × 3 1/8	8 13/16	2 1/16	6 9/16		323.8	3.3	192.8	2.5	99.9	29.37
		3/4	3 3/16 × 4 1/8 × 3 3/16	9	2 1/8	6 5/8		351.5	3.3	220.5	2.6	109.7	32.25
10	7" Web Same Thickness as Z-Bar	5/16	3 3/16 × 5 × 3 3/16	10 5/16	1 11/16	9 1/16		193.8	3.5	147.4	3.0	53.1	15.63
		3/8	3 1/4 × 5 1/16 × 3 1/4	10 1/2	1 3/4	9 1/8		231.0	3.5	183.4	3.1	64.0	18.83
		7/16	3 5/16 × 5 1/8 × 3 5/16	10 11/16	1 13/16	9 3/16		267.6	3.4	222.0	3.1	75.0	22.06
		1/2	3 1/4 × 5 × 3 1/4	10 1/2	1 7/8	8 3/4		287.6	3.4	234.4	3.1	83.0	24.42
		9/16	3 5/16 × 5 1/16 × 3 5/16	10 11/16	1 15/16	8 13/16		321.1	3.4	273.7	3.1	93.7	27.58
		5/8	3 3/8 × 5 1/8 × 3 3/8	10 7/8	2	8 7/8		354.3	3.3	315.6	3.2	104.7	30.78
		11/16	3 1/4 × 5 × 3 1/4	10 11/16	2 1/16	8 7/16		364.8	3.3	320.0	3.1	111.0	32.65
		3/4	3 5/16 × 5 1/16 × 3 5/16	10 7/8	2 1/8	8 1/2		395.5	3.3	363.0	3.1	121.7	35.81
12	8" Web Same Thickness as Z-Bar	3/8	3 1/2 × 6 × 3 1/2	12 3/8	1 3/4	11		337.0	3.9	287.8	3.6	72.6	21.36
		7/16	3 9/16 × 6 1/16 × 3 9/16	12 9/16	1 13/16	11 1/16		391.4	3.9	346.9	3.7	85.2	25.06
		1/2	3 5/8 × 6 1/8 × 3 5/8	12 3/4	1 7/8	11 1/8		444.6	3.9	409.2	3.7	97.7	28.76
		9/16	3 1/2 × 6 × 3 1/2	12 9/16	1 15/16	10 11/16		469.1	3.8	426.3	3.6	106.1	31.22
		5/8	3 9/16 × 6 1/16 × 3 9/16	12 3/4	2	10 3/4		518.0	3.8	489.2	3.7	118.4	34.84
		11/16	3 5/8 × 6 1/8 × 3 5/8	12 15/16	2 1/16	10 13/16		566.5	3.8	555.8	3.8	130.9	38.50
		3/4	3 1/2 × 6 × 3 1/2	12 3/4	2 1/8	10 3/8		579.7	3.7	562.4	3.7	137.9	40.56
		13/16	3 9/16 × 6 1/16 × 3 9/16	12 15/16	2 3/16	10 7/16		622.5	3.7	628.2	3.7	149.6	44.02
		7/8	3 5/8 × 6 1/8 × 3 5/8	13 1/8	2 1/4	10 1/2		666.6	3.7	699.1	3.8	162.0	47.64

TABLE 77.

Properties of Chord Sections.

McClintic-Marshal Construction Co. Standards.

Properties of Two Angles and One Web Plate.



Long Legs Turned Out. Top of Plate ¼" Below Backs of Angles.

Size of Web Plate.	Size of Angles.	Total Area.	Axis A-A.				Axis B-B.	
			Moment of Inertia.	Radius of Gyration.	Section Modulus.	Centroid.	Moment of Inertia.	Radius of Gyration.
		A	I_A	r_A	S_A	e	I_B	r_B
In.	In.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.
6×¼	2 ×2 ×¼	3.38	11.1	1.81	6.3	1.77	1.7	.70
	2½×2 ×¼	3.62	11.7	1.80	7.1	1.66	3.1	.93
7×¼	2 ×2 ×¼	3.63	17.1	2.17	9.1	1.87	1.7	.68
	2½×2 ×¼	3.87	17.8	2.14	8.9	1.99	3.1	.90
	3 ×2 ×¼	4.13	18.7	2.13	10.0	1.87	5.1	1.12
	3 ×2½×¼	4.37	18.7	2.07	9.9	1.90	5.2	1.09
8×¼	2 ×2 ×¼	3.88	24.4	2.51	9.8	2.48	1.7	.66
	2½×2 ×¼	4.12	25.6	2.49	10.9	2.34	3.1	.87
	2½×2½×¼	4.38	25.6	2.42	11.0	2.33	3.1	.84
	2½×2½×5/16	4.94	27.1	2.34	12.5	2.16	3.9	.89
	3 ×2 ×¼	4.38	26.8	2.47	12.1	2.21	5.1	1.09
	3 ×2½×¼	4.62	26.8	2.41	12.1	2.22	5.2	1.06
	3 ×2½×5/16	5.24	28.7	2.30	13.6	2.04	6.5	1.11
	3½×2½×¼	4.88	27.9	2.39	13.3	2.10	8.0	1.28
	3½×2½×5/16	5.56	29.2	2.29	15.1	1.94	10.1	1.35
8×5/16	2½×2½×5/16	5.44	31.7	2.41	13.5	2.35	4.1	.87
	3 ×2½×¼	5.12	31.3	2.47	12.9	2.42	5.3	1.02
	3 ×2½×5/16	5.74	33.2	2.40	14.8	2.24	6.7	1.08
	3½×2½×¼	5.38	32.6	2.46	14.2	2.30	8.2	1.24
	3½×2½×5/16	6.06	34.3	2.38	16.1	2.13	10.4	1.31
8×⅜	3½×2½×5/16	6.56	39.1	2.44	17.1	2.29	10.6	1.28
	4 ×3 ×5/16	7.18	40.6	2.38	18.1	2.22	15.2	1.47
	4 ×3 ×⅜	7.96	42.5	2.31	20.3	2.09	18.6	1.53
9×¼	2½×2½×¼	4.63	35.4	2.76	13.2	2.68	3.1	.82
	3 ×2 ×¼	4.63	37.3	2.84	13.5	2.77	5.1	1.05
	3 ×2½×¼	4.87	37.0	2.75	14.5	2.55	5.2	1.03
	3½×2½×¼	5.13	38.4	2.73	15.8	2.43	8.0	1.25
9×5/16	3 ×2½×5/16	6.05	45.8	2.75	17.5	2.62	6.7	1.05
	3½×2½×5/16	6.37	47.5	2.73	19.0	2.50	10.3	1.28
	3½×2½×⅜	7.03	49.5	2.65	21.1	2.34	12.4	1.33
	4 ×3 ×5/16	6.99	49.1	2.65	20.2	2.41	15.1	1.48
10×¼	2½×2½×¼	4.88	47.2	3.10	15.5	3.04	3.1	.80
	2½×2½×5/16	5.44	50.1	3.03	17.8	2.82	3.9	.85
	3 ×2 ×¼	4.88	49.3	3.19	16.8	2.93	5.1	1.03
	3 ×2½×¼	5.12	49.3	3.09	17.0	2.90	5.2	1.00
	3 ×2½×5/16	5.74	52.2	3.02	19.6	2.67	6.5	1.06
	3½×2½×¼	5.38	51.3	3.09	18.5	2.77	8.0	1.22
	3½×2½×5/16	6.06	54.0	2.99	21.2	2.55	10.1	1.29
	4 ×3 ×5/16	6.68	55.7	2.89	22.8	2.44	14.8	1.49
10×5/16	2½×2½×5/16	6.07	58.6	3.10	19.1	3.07	4.1	.82
	3 ×2½×¼	5.75	57.6	3.16	18.2	3.16	5.3	.96
	3 ×2½×5/16	6.37	61.2	3.10	21.0	2.91	6.7	1.02
	3½×2½×¼	6.01	60.0	3.16	19.8	3.03	8.2	1.17
	3½×2½×5/16	6.69	63.4	3.08	22.7	2.80	10.3	1.25
	4 ×3 ×5/16	7.31	65.5	2.99	24.3	2.69	15.1	1.44
	4 ×3 ×⅜	8.09	68.3	2.91	27.2	2.51	18.2	1.50
	5 ×3 ×5/16	7.93	69.2	2.96	27.8	2.49	28.7	1.91
	5 ×3 ×⅜	8.85	72.1	2.85	31.1	2.32	34.4	1.97
	5 ×3½×5/16	8.25	69.3	2.89	27.6	2.51	28.8	1.87
	5 ×3½×⅜	9.23	72.4	2.81	30.8	2.35	34.6	1.94
10×⅜	3 ×2½×5/16	6.99	69.5	3.15	22.2	3.13	6.9	.99
	3½×2½×5/16	7.31	72.1	3.14	23.9	3.01	10.6	1.21
	4 ×3 ×5/16	7.93	74.5	3.07	25.9	2.88	15.5	1.40
	4 ×3 ×⅜	8.71	77.8	2.99	28.7	2.71	18.6	1.46
	5 ×3 ×5/16	8.55	78.9	3.03	29.3	2.69	29.7	1.85
	5 ×3 ×⅜	9.47	82.4	2.94	32.8	2.51	35.1	1.93
12×¼	3 ×2½×¼	5.62	81.2	3.80	22.2	3.65	5.2	.96
	3 ×2½×5/16	6.24	86.2	3.73	25.6	3.37	6.5	1.02
	3½×2½×¼	5.88	84.3	3.78	24.2	3.49	8.0	1.17
	3½×2½×5/16	6.56	89.1	3.67	27.8	3.20	10.1	1.24
	4 ×3 ×5/16	7.18	92.0	3.58	30.2	3.05	14.8	1.44
	5 ×3 ×5/16	7.80	96.8	3.52	34.3	2.82	28.1	1.90
	5 ×3 ×⅜	8.72	100.8	3.41	38.6	2.61	33.8	1.97
	5 ×3½×5/16	8.12	96.8	3.45	34.0	2.85	28.3	1.87
	5 ×3½×⅜	9.10	100.6	3.33	38.1	2.64	33.9	1.94

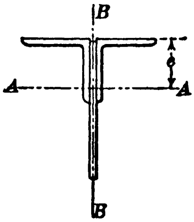
Note: Section modulus, S_A, is given for top fiber.

TABLE 77.—*Continued.*

PROPERTIES OF CHORD SECTIONS.

McCLINTIC-MARSHALL CONSTRUCTION CO. STANDARDS.

Properties of Two Angles and One Web Plate.



Long Legs Turned Out. Top of Plate ¼" Below Backs of Angles.

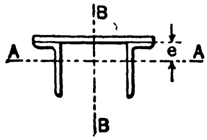
Size of Web Plate.	Size of Angles.	Total Area.	Axis A-A. Moment of Inertia.	Axis A-A. Radius of Gyration.	Axis A-A. Section Modulus.	Axis A-A. Centroid.	Axis B-B. Moment of Inertia.	Axis B-B. Radius of Gyration.
		A	I_A	r_A	S_A	e	I_B	r_B
In.	In.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.
$12 \times \frac{5}{16}$	$3 \times 2\frac{1}{2} \times \frac{1}{4}$	6.37	94.8	3.84	24.0	3.95	5.3	.92
	$3 \times 2\frac{1}{2} \times \frac{5}{16}$	6.99	100.7	3.79	27.5	3.67	6.7	.98
	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	6.63	98.5	3.86	25.9	3.81	8.2	1.11
	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	7.31	104.5	3.78	29.6	3.53	10.3	1.19
	$4 \times 3 \times \frac{5}{16}$	7.93	107.9	3.70	32.0	3.37	15.1	1.38
	$4 \times 3 \times \frac{3}{8}$	8.71	112.8	3.60	35.8	3.15	18.2	1.44
	$5 \times 3 \times \frac{5}{16}$	8.55	113.8	3.64	36.3	3.13	28.7	1.82
	$5 \times 3 \times \frac{3}{8}$	9.47	119.0	3.55	40.9	2.91	34.4	1.92
	$5 \times 3\frac{1}{2} \times \frac{5}{16}$	8.87	113.9	3.58	36.4	3.13	28.8	1.80
	$5 \times 3\frac{1}{2} \times \frac{3}{8}$	9.85	119.0	3.47	40.9	2.92	34.6	1.88
$12 \times \frac{3}{8}$	$3 \times 2\frac{1}{2} \times \frac{5}{16}$	7.74	114.3	3.84	29.3	3.91	6.9	.95
	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	8.06	118.5	3.83	31.4	3.77	10.6	1.15
	$4 \times 3 \times \frac{5}{16}$	8.68	122.7	3.76	34.0	3.61	15.5	1.34
	$4 \times 3 \times \frac{3}{8}$	9.46	128.4	3.68	38.0	3.38	18.6	1.40
	$5 \times 3 \times \frac{5}{16}$	9.30	129.9	3.74	38.4	3.38	29.2	1.77
	$5 \times 3 \times \frac{3}{8}$	10.22	135.8	3.64	43.2	3.14	35.1	1.85
	$5 \times 3\frac{1}{2} \times \frac{5}{16}$	9.62	129.5	3.80	38.4	3.37	29.4	1.75
	$5 \times 3\frac{1}{2} \times \frac{3}{8}$	10.60	135.8	3.58	43.1	3.15	35.3	1.82
	$6 \times 3\frac{1}{2} \times \frac{3}{8}$	11.34	141.8	3.54	47.9	2.96	59.6	2.30
	$6 \times 4 \times \frac{3}{8}$	11.72	145.0	3.52	48.7	2.98	59.6	2.26
$12 \times \frac{7}{16}$	$4 \times 3 \times \frac{7}{16}$	10.99	149.1	3.68	45.1	3.31	22.3	1.42
	$5 \times 3 \times \frac{3}{8}$	10.97	150.0	3.69	44.8	3.35	35.8	1.81
	$5 \times 3\frac{1}{2} \times \frac{3}{8}$	11.35	151.5	3.65	45.2	3.35	35.9	1.78
	$5 \times 3\frac{1}{2} \times \frac{7}{16}$	12.31	157.1	3.57	49.6	3.17	42.0	1.85
	$6 \times 3\frac{1}{2} \times \frac{3}{8}$	12.09	158.4	3.62	50.2	3.16	60.6	2.24
	$6 \times 3\frac{1}{2} \times \frac{7}{16}$	13.19	164.3	3.52	55.0	2.99	70.6	2.31
	$6 \times 4 \times \frac{7}{16}$	13.61	164.4	3.48	54.8	3.00	70.6	2.28

Size of Web Plate.	Size of Angles.	Total Area.	Axis A-A. Moment of Inertia.	Axis A-A. Radius of Gyration.	Axis A-A. Section Modulus.	Axis A-A. Centroid.	Axis B-B. Moment of Inertia.	Axis B-B. Radius of Gyration.
		A	I_A	r_A	S_A	e	I_B	r_B
In.	In.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.
$12 \times \frac{1}{2}$	$4 \times 3 \times \frac{1}{2}$	12.50	168.6	3.67	49.1	3.43	26.4	1.46
	$5 \times 3 \times \frac{3}{8}$	11.72	166.4	3.76	46.8	3.55	36.5	1.77
	$5 \times 3 \times \frac{1}{2}$	13.50	178.2	3.63	55.9	3.19	48.9	1.90
	$5 \times 3\frac{1}{2} \times \frac{1}{2}$	14.00	178.2	3.56	55.7	3.20	49.7	1.88
	$6 \times 3\frac{1}{2} \times \frac{3}{8}$	12.84	174.1	3.69	52.0	3.35	61.6	2.19
	$6 \times 3\frac{1}{2} \times \frac{1}{2}$	15.00	186.3	3.52	62.1	3.00	82.0	2.34
	$6 \times 4 \times \frac{1}{2}$	15.50	186.3	3.52	61.5	3.03	82.5	2.31
$14 \times \frac{3}{8}$	$4 \times 3 \times \frac{3}{8}$	10.21	196.5	4.39	47.8	4.11	18.6	1.35
	$5 \times 3 \times \frac{3}{8}$	10.97	207.4	4.34	54.1	3.83	35.1	1.79
	$5 \times 3\frac{1}{2} \times \frac{3}{8}$	11.35	207.5	4.28	54.4	3.81	35.3	1.76
	$6 \times 3\frac{1}{2} \times \frac{3}{8}$	12.09	216.6	4.23	60.5	3.59	59.6	2.22
	$6 \times 4 \times \frac{3}{8}$	12.47	216.7	4.16	60.5	3.59	59.6	2.19
$14 \times \frac{1}{2}$	$4 \times 3 \times \frac{1}{2}$	13.50	258.2	4.37	62.2	4.16	26.4	1.40
	$5 \times 3 \times \frac{1}{2}$	14.50	273.3	4.34	70.1	3.89	48.9	1.84
	$5 \times 3\frac{1}{2} \times \frac{1}{2}$	15.00	273.5	4.27	70.8	3.87	49.2	1.81
	$6 \times 3\frac{1}{2} \times \frac{3}{8}$	13.84	265.7	4.38	65.3	4.07	61.6	2.11
	$6 \times 3\frac{1}{2} \times \frac{1}{2}$	16.00	285.3	4.22	78.3	3.64	82.0	2.26
	$6 \times 4 \times \frac{1}{2}$	16.50	285.0	4.16	78.1	3.65	82.5	2.24
$16 \times \frac{3}{8}$	$5 \times 3\frac{1}{2} \times \frac{3}{8}$	12.10	299.6	4.98	66.4	4.52	35.3	1.70
	$6 \times 3\frac{1}{2} \times \frac{3}{8}$	12.84	312 6	4.94	73.3	4.27	59.7	2.16
	$6 \times 3\frac{1}{2} \times \frac{1}{2}$	15.00	334.7	4 72	88.1	3.80	80.0	2.30
$16 \times \frac{1}{2}$	$6 \times 3\frac{1}{2} \times \frac{3}{8}$	14.84	382.5	5.09	79.5	4.81	61.6	2.04
	$6 \times 3\frac{1}{2} \times \frac{7}{16}$	15.94	399.0	5.03	87.5	4.55	71.9	2.13
	$6 \times 3\frac{1}{2} \times \frac{1}{2}$	17.00	412.4	4.92	95.5	4.32	82.0	2.18
	$6 \times 4 \times \frac{1}{2}$	17.50	412.1	4.85	95.6	4.31	82.5	2.17

NOTE: Section modulus, S_A, is given for top fiber.

TABLE 78.

Properties of Top Chord Sections.



Properties of Two Angles and One Cover Plate. Angles Turned Out.

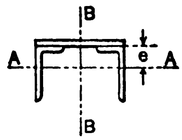
Short Legs Against Plate, and Turned Out. Edges of Angles Flush with Edges of Plate.

Series 1 and 2.	Series 1.								Series 2.							
			Axis A-A.				Axis B-B.				Axis A-A.				Axis B-B.	
Size of Plate.	Size of Angles.	Total Area.	Moment of Inertia.	Radius of Gyration.	Section Modulus, Upper Fiber.	Centroid.	Moment of Inertia.	Radius of Gyration.	Size of Angles.	Total Area.	Moment of Inertia.	Radius of Gyration.	Section Modulus, Upper Fiber.	Centroid.	Moment of Inertia.	Radius of Gyration.
		A	I_A	r_A	S_A	e	I_B	r_B		A	I_A	r_A	S_A	e	I_B	r_B
In.	In.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.	In.	In.²	In.⁴	In.	In.³	In.	In.⁴	In
10x¼	3x2½x¼	5.12	3.7	.86	5.8	.40	48.5	3.08	3x2½x⅜	6.34	5.1	.90	6.6	.53	62.5	3.14
	4x3 x¼	5.88	8.2	1.18	9.0	.66	49.0	2.89	4x3 x⅜	7.46	11.2	1.23	10.6	.81	63.0	2.91
10x5/16	3x2½x¼	5.74	4.0	.84	6.3	.33	53.7	3.05	3x2½x⅜	6.96	5.6	.90	7.3	.46	67.7	3.12
	4x3 x¼	6.50	8.7	1.16	10.0	.57	54.2	2.89	4x3 x⅜	8.08	11.9	1.22	11.5	.73	68.2	2.90
12x¼	3x2½x¼	5.62	3.9	.83	6.4	.36	82.8	3.84	3x2½x⅜	6.84	5.3	.89	7.4	.48	106.2	3.94
	4x3 x¼	6.38	8.5	1.16	10.2	.60	86.1	3.67	4x3 x⅜	7.96	11.6	1.21	11.7	.75	110.7	3.73
	5x3½x5/16	8.12	18.8	1.52	15.5	.96	98.6	3.48	5x3½x7/16	10.06	24.3	1.56	17.9	1.11	124.0	3.51
12x5/16	3x2½x¼	6.37	4.1	.80	6.9	.28	91.8	3.79	3x2½x⅜	7.59	5.7	.87	8.0	.41	115.2	3.89
	4x3 x¼	7.13	9.1	1.13	11.1	.51	95.1	3.65	4x3x ⅜	8.71	12.4	1.19	12.7	.66	119.7	3.71
	5x3½x5/16	8.87	19.8	1.49	17.1	.85	107.6	3.48	5x3½x7/16	10.81	25.6	1.54	19.4	1.01	133.0	3.51
12x⅜	3x2½x¼	7.12	4.4	.79	7.5	.22	100.8	3.76	3x2½x⅜	8.34	6.1	.86	8.6	.34	124.2	3.86
	4x3 x¼	7.88	9.5	1.10	11.9	.43	104.1	3.64	4x3 x⅜	9.46	13.0	1.18	13.8	.58	128.7	3.69
	5x3½x5/16	9.62	20.8	1.47	18.4	.76	116.6	3.48	5x3½x7/16	11.56	26.9	1.53	20.7	.92	142.0	3.50
14x¼	3x2½x¼	6.12	4.0	.81	7.0	.32	128.4	4.58	3x2½x⅜	7.34	5.5	.87	8.1	.44	163.5	4.72
	4x3x ¼	6.88	8.8	1.13	11.0	.55	135.9	4.45	4x3 x⅜	8.46	12.0	1.19	12.7	.70	174.3	4.54
	5x3½x5/16	8.62	19.3	1.50	17.0	.89	159.1	4.30	5x3½x7/16	10.56	25.0	1.54	19.2	1.05	199.8	4.35
	6x4 x⅜	10.72	37.1	1.36	24.4	1.27	179.1	4.09	6x4 x½	13.00	46.2	1.88	27.7	1.42	220.9	4.12
14x5/16	3x2½x¼	6.99	4.2	.78	7.7	.24	142.7	4.52	3x2½x⅜	8.21	5.9	.85	8.7	.37	177.7	4.65
	4x3 x¼	7.75	9.3	1.11	12.3	.45	150.2	4.40	4x3 x⅜	9.33	12.8	1.17	13.9	.61	188.6	4.49
	5x3½x5/16	9.49	20.4	1.47	18.7	.78	173.4	4.27	5x3½x7/16	11.43	26.4	1.52	20.9	.95	214.1	4.33
	6x4 x⅜	11.59	39.0	1.83	26.7	1.15	193.4	4.08	6x4 x½	13.87	48.6	1.87	30.0	1.31	235.1	4.11
14x⅜	3x2½x¼	7.87	4.5	.76	8.2	.18	157.0	4.47	3x2½x⅜	9.09	6.3	.83	9.4	.30	192.0	4.59
	4x3 x¼	8.63	10.2	1.07	13.1	.37	164.5	4.37	4x3 x⅜	10.21	13.5	1.15	14.8	.53	202.9	4.46
	5x3½x5/16	10.37	21.4	1.44	20.2	.69	187.7	4.25	5x3½x7/16	12.31	27.6	1.50	22.4	.86	228.4	4.31
	6x4 x⅜	12.47	40.8	1.81	28.7	1.04	207.7	4.08	6x4 x½	14.75	50.8	1.85	32.0	1.22	249.5	4.11
16x¼	4x3 x¼	7.38	9.0	1.10	12.0	.50	199.5	5.20	4x3 x⅜	8.96	12.3	1.18	13.8	.65	254.8	5.33
	5x3½x5/16	9.12	19.8	1.47	18.2	.84	236.8	5.09	5x3½x7/16	11.06	25.7	1.52	20.6	1.00	296.9	5.18
	6x4 x⅜	11.22	38.0	1.84	26.2	1.20	271.3	4.91	6x4 x½	13.50	47.4	1.87	27.4	1.36	334.4	4.98
16x5/16	4x3 x¼	8.38	9.5	1.07	13.2	.41	220.9	5.13	4x3 x⅜	9.96	13.1	1.15	15.1	.56	276.2	5.27
	5x3½x5/16	10.12	20.9	1.44	20.1	.73	258.1	5.05	5x3½x7/16	12.06	27.1	1.50	22.6	.89	318.2	5.14
	6x4 x⅜	12.22	42.0	1.81	28.8	1.08	292.7	4.90	6x4 x½	14.50	49.9	1.85	32.0	1.25	355.7	4.95
16x⅜	5x3½x5/16	11.12	21.9	1.40	21.8	.63	279.4	5.02	5x3½x7/16	13.06	28.5	1.48	24.4	.80	339.6	5.10
	6x4 x⅜	13.22	41.9	1.78	31.0	.98	314.0	4.87	6x4 x½	15.50	52.2	1.83	34.3	1.15	377.0	4.93
	8x6 x7/16	17.86	106.0	2.44	54.7	1.56	307.8	4.15	8x6 x9/16	21.12	129.6	2.48	61.4	1.74	361.3	4.13

TABLE 79.

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Two Angles and One Cover Plate. Angles Turned In.



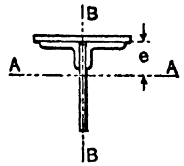
Short Legs Against Plate, and Turned In. Backs of Angles Flush with Edges of Plate.

Series 1 and 2.	Series 1.								Series 2.							
			Axis A-A.				Axis B-B.				Axis A-A.				Axis B-B.	
Size of Plate.	Size of Angles.	Total Area.	Moment of Inertia.	Radius of Gyration.	Section Modulus, Upper Fiber.	Centroid.	Moment of Inertia.	Radius of Gyration.	Size of Angles.	Total Area.	Moment of Inertia.	Radius of Gyration.	Section Modulus, Upper Fiber.	Centroid.	Moment of Inertia.	Radius of Gyration.
		A	I_A	r_A	S_A	e	I_B	r_B		A	I_A	r_A	S_A	e	I_B	r_B
In.	In.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.	In.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.
8x¼	3x2½x¼	4.62	3.6	0.88	5.1	.46	41.4	2.99	3x2½x⅜	5.84	4.9	.91	5.8	.59	54.3	3.05
"	4x3 x¼	5.38	7.9	1.21	8.1	.73	49.4	3.03	4x3 x⅜	6.96	10.8	1.25	9.6	.88	66.0	3.08
8x5/16	3x2½x¼	5.12	3.9	0.87	5.6	.39	44.0	2.93	3x2½x⅜	6.34	5.3	.91	6.4	.52	57.0	3.00
"	4x3 x¼	5.88	8.4	1.20	8.7	.65	52.1	2.98	4x3 x⅜	7.46	11.4	1.24	10.3	.80	68.6	3.03
10x¼	3x2½x¼	5.12	3.8	0.86	5.8	.41	71.7	3.74	3x2½x⅜	6.34	5.2	.90	6.6	.53	93.6	3.84
"	4x3 x¼	5.88	8.4	1.19	9.2	.66	85.0	3.80	4x3 x⅜	7.46	11.3	1.23	10.6	.81	113.0	3.89
"	5x3½x5/16	7.62	18.1	1.54	14.1	1.03	114.9	3.88	5x3½x7/16	9.56	23.5	1.57	16.5	1.17	147.9	3.93
"	6x4 x⅜	9.72	34.9	1.89	21.0	1.41	149.6	3.92	6x4 x½	12.00	43.7	1.91	24.3	1.55	186.1	3.94
10x5/16	3x2½x¼	5.74	4.1	0.83	6.2	.33	76.9	3.66	3x2½x⅜	6.96	5.6	.90	7.3	.46	98.8	3.76
"	4x3 x¼	6.50	8.8	1.16	10.0	.57	90.2	3.72	4x3 x⅜	8.08	12.0	1.22	11.5	.73	118.2	3.82
"	5x3½x5/16	8.24	19.2	1.53	15.5	.93	120.1	3.82	5x3½x7/16	10.18	24.7	1.56	17.8	1.08	153.2	3.88
"	6x4 x⅜	10.34	36.7	1.88	22.6	1.31	154.9	3.87	6x4 x½	12.62	45.6	1.90	25.8	1.46	191.3	3.89
10x⅜	3x2½x¼	6.37	4.2	0.81	6.6	.26	82.1	3.59	3x2½x⅜	7.59	5.9	.88	7.7	.39	104.0	3.70
"	4x3 x¼	7.13	9.3	1.14	10.6	.49	95.4	3.66	4x3 x⅜	8.71	12.6	1.20	12.2	.66	123.4	3.76
"	5x3½x5/16	8.87	22.0	1.50	16.5	.84	125.4	3.76	5x3½x7/16	10.81	25.9	1.54	18.8	1.00	158.4	3.83
"	6x4 x⅜	10.97	38.2	1.87	24.0	1.21	160.0	3.82	6x4 x½	13.25	47.5	1.89	27.3	1.37	196.5	3.85
12x¼	4x3 x¼	6.38	8.6	1.16	10.2	.60	132.3	4.55	4x3 x⅜	7.96	11.7	1.21	11.7	.75	175.0	4.69
"	5x3½x5/16	8.12	18.8	1.52	15.5	.96	177.8	4.68	5x3½x7/16	10.06	24.3	1.56	17.9	1.11	228.4	4.76
"	6x4 x⅜	10.22	36.0	1.88	22.8	1.33	230.6	4.76	6x4 x½	12.50	45.0	1.90	26.0	1.48	287.0	4.79
12x5/16	4x3 x¼	7.13	9.1	1.13	11.1	.51	141.3	4.45	4x3 x⅜	8.71	12.4	1.19	12.7	.66	184.0	4.60
"	5x3½x5/16	8.87	19.8	1.49	17.1	.85	186.8	4.59	5x3½x7/16	10.81	25.6	1.54	19.4	1.01	237.6	4.69
"	6x4 x⅜	10.97	37.9	1.86	24.8	1.22	239.6	4.67	6x4 x½	13.25	47.2	1.89	27.9	1.38	296.0	4.73
12x⅜	4x3 x¼	7.88	9.5	1.10	11.9	.43	150.3	4.37	4x3 x⅜	9.46	13.1	1.18	13.8	.58	193.0	4.52
"	5x3½x5/16	9.62	20.8	1.47	18.4	.76	195.8	4.51	5x3½x7/16	11.56	26.9	1.53	20.7	.92	246.6	4.62
"	6x4 x⅜	11.72	39.6	1.84	26.4	1.12	248.6	4.61	6x4 x½	14.00	49.2	1.87	29.6	1.29	305.0	4.67
14x¼	4x3 x¼	6.88	8.8	1.13	11.0	.55	192.4	5.29	4x3 x⅜	8.46	12.0	1.19	12.7	.70	252.9	5.47
"	5x3½x5/16	8.62	19.3	1.50	17.0	.89	257.0	5.46	5x3½x7/16	10.56	25.0	1.54	19.2	1.05	328.9	5.58
"	6x4 x⅜	10.72	37.1	1.86	24.4	1.27	332.2	5.56	6x4 x½	13.00	46.2	1.88	27.7	1.42	412.9	5.63
14x5/16	4x3 x¼	7.75	9.3	1.11	12.3	.45	206.7	5.16	4x3 x⅜	9.33	12.8	1.17	13.9	.61	267.2	5.34
"	5x3½x5/16	9.49	20.4	1.47	18.7	.78	271.3	5.34	5x3½x7/16	11.43	26.4	1.52	20.9	95	343.1	5.48
"	6x4 x⅜	11.59	39.0	1.83	26.7	1.15	346.4	5.46	6x4 x½	13.87	48.6	1.87	30.0	1.31	427.2	5.54
14x⅜	4x3 x¼	8.63	9.9	1.07	13.1	.37	221.0	5.06	4x3 x⅜	10.21	13.5	1.15	14.8	.53	281.5	5.25
"	5x3½x5/16	10.37	21.4	1.44	20.2	.69	285.5	5.24	5x3½x7/16	12.31	27.6	1.50	22.4	.86	357.4	5.39
"	6x4 x⅜	12.47	40.8	1.81	28.9	1.04	360.6	5.38	6x4 x½	14.75	50 8	1.85	32.0	1.22	441.4	5.47
"	8x6 x7/16	17.11	103.7	2.46	51.6	1.64	489.7	5.35	8x6 x9/16	20.37	126.7	2.49	58 1	1.81	591.2	5.40

TABLE 80.

Properties of Top Chord Sections.

Properties of Two Angles, One Web Plate and One Cover Plate.



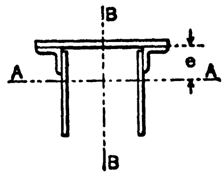
Long Legs Turned Out. Top of Web Plate ¼″ Below Backs of Angles.

Series 1 and 2.		Series 1.								Series 2.							
				Axis A–A.				Axis B–B.				Axis A–A.				Axis B–B.	
Size of Web Plate.	Size of Angles.	Size of Top Plate.	Total Area.	Moment of Inertia	Radius of Gyration.	Section Modulus, Upper Fiber.	Centroid.	Moment of Inertia.	Radius of Gyration.	Size of Top Plate.	Total Area.	Moment of Inertia.	Radius of Gyration.	Section Modulus, Upper Fiber.	Centroid.	Moment of Inertia.	Radius of Gyration.
			A	I_A	r_A	S_A	e	I_B	r_B		A	I_A	r_A	S_A	e	I_B	r_B
In.	In.	In.	In.^{2}	In.^{4}	In.	In.^{3}	In.	In.^{4}	In.	In.	In.^{2}	In.^{4}	In.	In.^{3}	In.	In.^{4}	In.
6x¼	2 x2 x¼	6x¼	4.88	14.8	1.74	10.3	1.19	6.1	1.12	6x⅜	5.63	16.2	1.70	11.8	.99	8.4	1.22
8x¼	2 x2 x¼	6x¼	5.38	31.6	2.42	15.8	1.75	6.1	1.07	6x⅜	6.13	34.5	2.37	18.4	1.50	8.4	1.17
	2½x2½x¼	6x¼	5.88	32.3	2.34	16.5	1.71	7.6	1.14	6x⅜	6.63	35.0	2.30	18.9	1.48	9.9	1.22
	" 5/16	6x¼	6.44	32.9	2.26	17.5	1.63	8.4	1.14	6x⅜	7.19	35.5	2.22	19.7	1.43	10.7	1.22
	3 x2½x¼	8x¼	6.62	34.4	2.28	19.5	1.51	15.8	1.55	8x⅜	7.62	37.1	2 21	22.5	1.27	21.2	1.67
	" 5/16	8x¼	7.24	35.3	2.21	20.8	1.45	17.1	1.54	8x⅜	8.24	37.7	2.14	23.5	1.23	22.5	1 65
8x5/16	2½x2½x¼	6x¼	6.38	38.0	2.44	17.6	1.91	7.6	1.10	6x⅜	7.13	41.3	2.41	20.2	1.67	10.0	1.18
	" 5/16	6x¼	6.94	38.9	2.37	18.8	1.82	8.4	1.10	6x⅜	7.69	41.9	2.33	21.1	1.61	10.8	1.18
	3 x2½x¼	8x¼	7.12	40.5	2.38	20.8	1.70	16.0	1.49	8x⅜	8.12	44.0	2.33	24.1	1.45	21.3	1.62
	" 5/16	8x¼	7.74	41.4	2.31	22.0	1.63	17.3	1.49	8x⅜	8.74	44.7	2.26	25.2	1.40	22.7	1.61
8x⅜	3 x2½x¼	8x¼	7.62	46.3	2.46	21.8	1.87	16.2	1.46	8x⅜	8.62	48.5	2.37	23.0	1.73	21.5	1.58
	" 5/16	8x¼	8.24	47.3	2.39	23.3	1.78	17.6	1 46	8x⅜	9.24	49.4	2.31	24.1	1.67	22.9	1.57
	4 x3 x5/16	10x⅜	10.93	54.9	2.24	31.1	1.40	46.8	2.07	10x½	12.18	58.6	2.19	34.3	1.21	57.2	2.17
	" ⅜	10x⅜	11.71	55.5	2.18	32.1	1.36	49.9	2.06	10x½	12.96	59.2	2.14	35.1	1.19	60.3	2.16
10x¼	2½x2½x¼	6x¼	6.38	58.1	3.06	23.0	2.30	7.6	1.09	6x⅜	7.13	63.4	2.96	26.1	2.02	9.9	1.18
	" 5/16	6x¼	6.94	60.0	2.94	24.7	2.18	8.4	1.10	6x⅜	7.69	64.4	2.89	27.9	1.93	10.7	1.18
	3 x2½x¼	8x¼	7.12	62.4	2.96	27.1	2.05	15.8	1.49	8x⅜	8.12	67.2	2.88	31.5	1.76	21.2	1.62
	" 5/16	8x¼	7.74	64.3	2.88	29.0	1.94	17.1	1.49	8x⅜	8.74	68.3	2.81	33.0	1.70	22.5	1.60
	4 x3 x5/16	10x⅜	10.43	72.4	2.63	38.6	1.50	46.1	2.10	10x½	11.68	76.5	2.56	42.7	1.29	56.5	2.20
	" ⅜	10x⅜	11.21	73.0	2.55	41.1	1.45	49.9	2.09	10x½	12.46	77.0	2.49	43.7	1.26	59.4	2.18
10x5/16	3 x2½x¼	8x¼	7.75	73.5	3.08	28.7	2.31	16.0	1.43	8x⅜	8.75	79.6	2.99	32.9	2.01	21.3	1.56
	" 5/16	8x¼	8 37	75.3	3.00	30.8	2.20	17.4	1 43	8x⅜	9.37	80.9	2.95	35.3	1.94	22.7	1.56
	4 x3 x5/16	10x⅜	11.06	85.8	2.79	41.1	1.71	46.4	2.06	10x½	12.31	91.0	2.75	46.7	1.49	56.9	2.15
	" ⅜	10x⅜	11.84	87.0	2.71	42.8	1.66	49.4	2.05	10x½	13.09	91.8	2.69	48.5	1.45	59.9	2.14
	5 x3½x5/16	12x⅜	12.75	90.5	2.66	46.8	1.56	82.8	2.56	12x½	14.25	95.8	2.59	51.8	1.35	100.8	2.66
	" ⅜	12x⅜	13.73	91.9	2.59	49.0	1.50	88.6	2.55	12x½	15.23	96.9	2.52	53.0	1.33	106.6	2.64
10x⅜	3 x2½x¼	8x¼	8.37	83.7	3.16	30.1	2.53	16.2	1.38	8x⅜	9.37	90.8	3.11	34.9	2.23	21.5	1.51
	" 5/16	8x¼	8.99	85.8	3.10	32.4	2.42	17.6	1.40	8x⅜	9.99	92.5	3.05	36.9	2.15	22.9	1.51
	4 x3 x5/16	10x⅜	11.68	98.4	2.92	43.2	1.90	46.8	2.00	10x½	12.93	104.6	2.81	47.3	1.67	57.2	2.10
	" ⅜	10x⅜	12.46	99.7	2.83	45.2	1.84	49.9	2.00	10x½	13.71	105.4	2.77	49.5	1.63	60.3	2.10
	5 x3½x5/16	12x⅜	13.37	103.7	2.78	49.4	1.73	83.4	2.50	12x½	14.87	110.0	2.72	54.7	1.51	101.4	2.61
	" ⅜	12x⅜	14.35	105.3	2.71	51.4	1.68	89.3	2.50	12x½	15.85	111.5	2.65	56.4	1.48	107.3	2.60

TABLE 81.

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Two Angles, Two Web Plates and One Cover Plate.



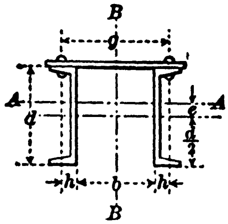
Angle Legs Turned Out. Edges of Angles Flush with Edges of Top Plate. Web Plates ¼" Below Backs of Angles.

Series 1 and 2.		Series 1.								Series 2.							
				Axis A-A.				Axis B-B.				Axis A-A.				Axis B-B.	
Size of Web Plates.	Size of Angles.	Size of Top Plate.	Total Area.	Moment of Inertia.	Radius of Gyration.	Section Modulus, Upper Fiber.	Centroid.	Moment of Inertia.	Radius of Gyration.	Size of Top Plate.	Total Area.	Moment of Inertia.	Radius of Gyration.	Section Modulus, Upper Fiber.	Centroid.	Moment of Inertia.	Radius of Gyration.
			A	I_A	r_A	S_A	e	I_B	r_B		A	I_A	r_A	S_A	e	I_B	r_B
In.	In.	In.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.	In.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.
8x¼	2½x2½x¼	10x¼	8.88	58	2.56	25.1	2.07	69.5	2.80	10x⅜	10.13	64	2.52	29.9	1.78	79.9	2.81
" ⅜	"	"	10.88	76	2.64	27.9	2.47	79.9	2.71	"	12.13	86	2.66	33.3	2.19	90.3	2.73
" ¼	2½x2½x⅜	"	9.96	60	2.45	27.4	1.94	82.2	2.87	"	11.21	66	2.43	31.8	1.69	92.6	2.87
" ⅜	"	"	11.96	80	2.58	30.8	2.33	91.6	2.77	"	13.21	88	2.57	35.6	2.08	102.1	2.78
8x¼	2½x2½x¼	12x¼	9.38	60	2.53	27.5	1.95	125.4	3.66	12x⅜	10.88	67	2.48	33.3	1.64	143.4	3.63
" ⅜	"	"	11.38	80	2.65	30.7	2.36	145.7	3.57	"	12.88	89	2.63	36.8	2.05	163.7	3.56
" ¼	2½x2½x⅜	"	10.46	62	2.44	29.7	1.84	146.4	3.74	"	11.96	68	2.39	35.0	1.57	164.4	3.71
" ⅜	"	"	12.46	83	2.58	33.3	2.23	166.7	3.65	"	13.96	91	2.56	39.2	1.95	184.7	3.64
10x¼	2½x2½x¼	12x¼	10.38	109	3.24	37.5	2.66	136.8	3.63	12x⅜	11.88	120	3.18	45.3	2.28	154.8	3.61
" ⅜	"	"	12.88	143	3.33	41.9	3.16	162.2	3.55	"	14.38	159	3.33	50.1	2.80	180.2	3.54
" ¼	2½x2½x⅜	"	11.46	113	3.14	41.2	2.49	157.8	3.71	"	12.96	123	3.03	48.4	2.17	175.8	3.68
" ⅜	"	"	13.96	149	3.27	46.1	2.98	183.2	3.62	"	15.46	164	3.25	53.9	2.66	201.2	3.61
10x¼	2½x2½x¼	14x¼	10.88	113	3.22	40.5	2.53	219.1	4.47	14x⅜	12.63	125	3.14	49.6	2.14	247.8	4.43
" ⅜	"	"	13.38	149	3.34	45.3	3.04	262.9	4.43	"	15.13	166	3.31	54.8	2.65	291.6	4.39
" ¼	2½x2½x⅜	"	11.96	116	3.12	43.9	2.38	250.6	4.58	"	13.71	127	3.04	52.7	2.04	279.2	4.51
" ⅜	"	"	14.46	154	3.26	46.3	2.88	294.4	4.51	"	16.21	170	3.23	58.3	2.53	323.0	4.46
12x⅜	3x3x¼	14x¼	15.38	244	3.98	60.4	3.79	258.1	4.10	14x⅜	17.13	270	3.97	72.2	3.37	286.7	4.09
" ½	"	"	18.38	295	4.01	66.5	4.19	296.1	4.01	"	20.13	328	4.03	78.5	3.80	324.8	4.02
" ⅜	3x3x⅜	"	16.72	254	3.90	66.7	3.56	292.6	4.18	"	18.47	279	3.88	77.9	3.20	321.2	4.17
" ½	"	"	19.72	309	3.96	73.2	3.97	330.7	4.09	"	21.47	339	3.97	84.7	3.62	359.3	4.09
12x⅜	3x3x¼	16x⅜	17.88	280	3.96	77.7	3.22	437.3	4.94	16x½	19.88	304	3.91	90.6	2.85	480.0	4.91
" ½	"	"	20.88	339	4.03	84.3	3.65	499.9	4.89	"	22.88	370	4.02	97.4	3.30	542.6	4.87
" ⅜	3x3x⅜	"	19.22	286	3.86	83.5	3.06	486.3	5.03	"	21.22	309	3.82	95.7	2.73	529.0	4.99
" ½	"	"	22.22	348	3.96	90.1	3.50	548.9	4.97	"	24.22	377	3.95	102.8	3.17	591.6	4.94
14x⅜	3x3x⅜	16x⅜	20.72	431	4.56	103.2	3.80	521.1	5.01	16x½	22.72	464	4.52	118.1	3.43	563.8	4.98
" ½	"	"	24.22	524	4.65	112.1	4.30	594.0	4.95	"	26.22	565	4.64	127.3	3.94	636.7	4.93
" ⅜	3x3x½	"	22.00	441	4.48	109.9	3.64	569.0	5.08	"	24.00	472	4.44	124.1	3.31	621.7	5.09
" ½	"	"	25.50	537	4.59	119.1	4.13	641.9	5.02	"	27.50	577	4.58	133.8	3.81	684.6	4.99
14x⅜	3x3x⅜	18x⅜	21.47	443	4.54	109.7	3.66	740.9	5.87	18x½	23.72	477	4.49	126.5	3.28	801.6	5.81
" ½	"	"	24.97	539	4.64	118.6	4.17	849.1	5.83	"	27.22	582	4.63	136.0	3.79	909.8	5.78
" ⅜	3x3x½	"	22.75	452	4.46	116.1	3.52	805.6	5.95	"	25.00	484	4.40	132.2	3.16	866.3	5.89
" ½	"	"	26.25	551	4.58	125.6	4.02	913.8	5.90	"	28.50	593	4.56	142.4	3.66	974.5	5.85

TABLE 82.

Properties of Top Chord Sections.

Properties of Top Chord Sections.



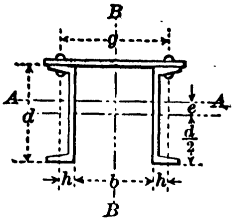
Two Channels and One Plate.

Section Number.	Channels. Depth.	Channels. Weight.	Cover Plate.	B to B Channels. b	Total Area.	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B	Gages. Plate. g	Gages. Channels. h	Web of Channels.	Max. Rivet.
	In.	Lb.	In.	In.	In.²	In.	In.⁴	In.⁴	In.	In.	In.	In.	In.	In.
1	5	6.7	8×¼	3⅞	5.90	0.89	23.9	34.7	2.01	2.42	6	1 1/16	.19	½
2			8×5/16	"	6.40	1.04	25.6	37.3	2.00	2.41	"	"	"	"
3			10×¼	5⅞	6.40	1.03	25.3	67.6	1.99	3.25	8	"	"	"
4			10×5/16	"	7.03	1.18	27.1	72.8	1.96	3.22	"	"	"	"
5			12×¼	7⅞	6.90	1.14	26.5	113.5	1.96	4.05	10	"	"	"
6			12×5/16	"	7.65	1.30	28.3	122.5	1.92	4.00	"	"	"	"
7	5	9.00	8×¼	3⅝	7.26	0.72	27.8	39.9	1.95	2.34	6	1 3/16	.33	½
8			8×5/16	"	7.76	0.84	29.7	42.5	1.95	2.33	"	"	"	"
9			10×¼	5⅝	7.76	0.84	29.5	79.7	1.95	3.20	8	"	"	"
10			10×5/16	"	8.39	0.99	31.7	84.9	1.94	3.17	"	"	"	"
11			12×¼	7⅝	8.26	0.95	31.0	135.1	1.93	4.04	10	"	"	"
12			12×5/16	"	9.01	1.10	33.3	144.1	1.92	3.99	"	"	"	"
13	6	8.2	10×¼	5½	7.26	1.08	42.0	73.1	2.41	3.17	7¾	1⅛	.20	⅝
14			10×5/16	"	7.89	1.25	44.8	78.3	2.38	3.15	"	"	"	"
15			12×¼	7½	7.76	1.21	44.0	124.0	2.38	4.00	9¾	"	"	"
16			12×5/16	"	8.51	1.39	46.9	133.0	2.35	3.95	"	"	"	"
17			14×5/16	9½	9.14	1.51	48.7	204.9	2.31	4.74	11¾	"	"	"
18			14×⅜	"	10.01	1.67	51.3	219.2	2.26	4.67	"	"	"	"
19	6	10.50	10×¼	5¼	8.64	0.90	47.6	83.1	2.34	3.09	7¾	1¼	.32	⅝
20			10×5/16	"	9.27	1.06	50.9	88.3	2.34	3.08	"	"	"	"
21			12×¼	7¼	9.14	1.02	50.0	143.0	2.33	3.95	9¾	"	"	"
22			12×5/16	"	9.89	1.19	53.5	152.0	2.32	3.91	"	"	"	"
23			14×5/16	9¼	10.52	1.31	55.8	235.7	2.30	4.73	11¾	"	"	"
24			14×⅜	"	11.39	1.47	58.9	250.3	2.27	4.68	"	"	"	"
25	7	9.8	10×¼	5¼	8.20	1.11	65.1	80.1	2.82	3.13	7¾	1¼	.21	⅝
26			10×5/16	"	8.83	1.30	69.2	85.3	2.80	3.11	"	"	"	"
27			12×¼	7¼	8.70	1.25	68.0	137.1	2.80	3.97	9¾	"	"	"
28			12×5/16	"	9.45	1.45	72.5	146.1	2.77	3.93	"	"	"	"
29			14×5/16	9¼	10.08	1.59	75.3	225.8	2.73	4.73	11¾	"	"	"
30			14×⅜	"	10.95	1.77	79.3	240.1	2.69	4.68	"	"	"	"
31	7	12.25	10×¼	5⅛	9.66	0.93	72.8	92.1	2.74	3.08	7¾	1 5/16	.32	⅝
32			10×5/16	"	10.29	1.11	77.5	97.3	2.74	3.07	"	"	"	"
33			12×¼	7⅛	10.16	1.07	76.2	159.1	2.73	3.95	9¾	"	"	"
34			12×5/16	"	10.91	1.25	81.4	168.1	2.73	3.92	"	"	"	"
35			14×5/16	9⅛	11.54	1.38	84.8	260.7	2.71	4.74	11¾	"	"	"
36			14×⅜	"	12.41	1.55	89.8	275.0	2.69	4.70	"	"	"	"

TABLE 82.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



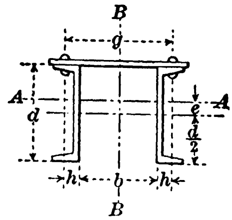
Two Channels and One Plate.

Section Number.	Channels. Depth.	Channels. Weight.	Cover Plate.	B to B Channels. b	Gross Area.	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B	Gages. Plate. g	Gages. Channel. h	Web of Channel.	Max. Rivet.
	In.	Lb.	In.	In.	In.^{2}	In.	In.^{4}	In.^{4}	In.	In.	In.	In.	In.	In.
37	8	11.5	12×1/4	7	9.70	1.28	99.9	150.2	3.21	3.93	9 1/2	1 1/4	.22	3/4
38			12×5/16	"	10.45	1.49	106.2	159.3	3.19	3.90	"	"	"	"
39			14×5/16	9	11.08	1.64	110.4	247.2	3.16	4.72	11 1/2	"	"	"
40			14×3/8	"	11.95	1.84	116.3	261.4	3.12	4.67	"	"	"	"
41			16×3/8	11	12.70	1.98	120.2	378.5	3.08	5.46	13 1/2	"	"	"
42			16×7/16	"	13.70	2.16	125.4	400.0	3.03	5.40	"	"	"	"
43	8	13.75	12×1/4	6 7/8	11.04	1.12	109.2	168.3	3.14	3.90	9 1/2	1 5/16	.31	3/4
44			12×5/16	"	11.79	1.32	116.3	177.3	3.13	3.87	"	"	"	"
45			14×5/16	8 7/8	12.42	1.46	121.0	276.6	3.12	4.71	11 1/2	"	"	"
46			14×3/8	"	13.29	1.65	127.8	290.9	3.10	4.67	"	"	"	"
47			16×3/8	10 7/8	14.04	1.78	132.5	421.9	3.07	5.48	13 1/2	"	"	"
48			16×7/16	"	15.04	1.96	138.7	443.2	3.03	5.42	"	"	"	"
49	9	13.4	12×1/4	6 3/4	10.78	1.29	140.9	162.9	3.62	3.89	9 1/2	1 3/8	.23	3/4
50			12×5/16	"	11.53	1.51	149.5	171.9	3.60	3.86	"	"	"	"
51			14×5/16	8 3/4	12.16	1.68	155.3	268.2	3.57	4.70	11 1/2	"	"	"
52			14×3/8	"	13.03	1.89	163.5	282.4	3.54	4.66	"	"	"	"
53			16×3/8	10 3/4	13.78	2.04	169.1	409.9	3.50	5.45	13 1/2	"	"	"
54			16×7/16	"	14.78	2.23	176.8	431.3	3.46	5.40	"	"	"	"
55	9	15.00	12×1/4	6 5/8	11.78	1.17	149.7	174.1	3.56	3.84	9 1/2	1 7/16	.29	3/4
56			12×5/16	"	12.53	1.39	158.8	183.1	3.55	3.82	"	"	"	"
57			14×5/16	8 5/8	13.16	1.54	165.2	287.4	3.54	4.67	11 1/2	"	"	"
58			14×3/8	"	14.03	1.75	174.2	301.7	3.52	4.63	"	"	"	"
59			16×3/8	10 5/8	14.78	1.90	180.3	439.4	3.49	5.44	13 1/2	"	"	"
60			16×7/16	"	15.78	2.09	188.6	460.7	3.45	5.40	"	"	"	"
61	10	15.3	14×5/16	8 1/2	13.30	1.70	211.7	289.4	3.99	4.67	11 1/2	1 1/2	.24	3/4
62			14×3/8	"	14.17	1.92	222.8	303.6	3.97	4.63	"	"	"	"
63			16×3/8	10 1/2	14.92	2.09	230.4	441.9	3.93	5.44	13 1/2	"	"	"
64			16×7/16	"	15.92	2.30	240.6	463.9	3.89	5.39	"	"	"	"
65			18×7/16	12 1/2	16.80	2.45	247.7	641.2	3.84	6.18	15 1/2	"	"	"
66			18×1/2	"	17.92	2.64	257.1	671.6	3.79	6.12	"	"	"	"
67	10	20.00	14×5/16	8 1/4	16.10	1.40	242.1	341.2	3.88	4.60	11 1/2	1 5/8	.38	3/4
68			14×3/8	"	16.97	1.60	255.2	355.0	3.87	4.57	"	"	"	"
69			16×3/8	10 1/4	17.72	1.75	264.4	520.4	3.86	5.41	13 1/2	"	"	"
70			16×7/16	"	18.72	1.95	276.9	542.0	3.84	5.37	"	"	"	"
71			18×7/16	12 1/4	19.60	2.09	286.9	752.3	3.82	6.19	15 1/2	"	"	"
72			18×1/2	"	20.72	2.28	297.8	782.7	3.79	6.14	"	"	"	"
73	10	25.00	14×5/16	7 7/8	19.04	1.18	271.8	383.9	3.77	4.48	11 1/2	1 13/16	.53	3/4
74			14×3/8	"	19.91	1.37	286.2	398.2	3.79	4.47	"	"	"	"
75			16×3/8	9 7/8	20.66	1.50	296.8	588.8	3.79	5.33	13 1/2	"	"	"
76			16×7/16	"	21.66	1.62	313.6	610.1	3.80	5.30	"	"	"	"
77			18×7/16	11 7/8	22.54	1.73	325.2	851.4	3.79	6.14	15 1/2	"	"	"
78			18×1/2	"	23.66	1.99	336.0	881.8	3.77	6.10	"	"	"	"

TABLE 82.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



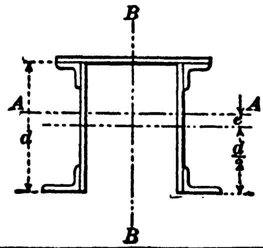
Two Channels and One Plate.

Section Number.	Channels. Depth.	Channels. Weight.	Cover Plate.	B to B Channels. b	Total Area.	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B	Gages. Plate. g	Gages. Channels. h	Web of Channels.	Max. Rivet.
	In.	Lb.	In.	In.	In.^{2}	In.	In.^{4}	In.^{4}	In.	In.	In.	In.	In.	In.
79	12	20.7	$16\times\frac{3}{8}$	$9\frac{3}{8}$	18.06	2.06	409.8	485.8	4.76	5.19	13	$1\frac{13}{16}$	.28	$\frac{7}{8}$
80			$16\times\frac{7}{16}$	"	19.06	2.28	427.6	507.1	4.74	5.16	"	"	"	"
81			$18\times\frac{3}{8}$	$11\frac{3}{8}$	18.91	2.21	422.4	682.1	4.73	6.00	15	"	"	"
82			$18\times\frac{7}{16}$	"	19.94	2.46	440.6	712.4	4.70	5.98	"	"	"	"
83			$20\times\frac{7}{16}$	$13\frac{3}{8}$	20.81	2.62	452.5	957.5	4.66	6.78	17	"	"	"
84			$20\times\frac{1}{2}$	"	22.06	2.83	469.8	999.1	4.61	6.73	"	"	"	"
85	12	25.00	$16\times\frac{3}{8}$	$9\frac{1}{4}$	20.64	1.79	451.4	550.0	4.67	5.16	13	$1\frac{7}{8}$	.39	$\frac{7}{8}$
86			$16\times\frac{7}{16}$	"	21.64	2.01	471.5	571.3	4.66	5.13	"	"	"	"
87			$18\times\frac{3}{8}$	$11\frac{1}{4}$	21.39	1.95	465.1	774.9	4.66	6.01	15	"	"	"
88			$18\times\frac{7}{16}$	"	22.52	2.17	486.5	805.2	4.64	5.98	"	"	"	"
89			$20\times\frac{7}{16}$	$13\frac{1}{4}$	23.39	2.32	500.3	1084.7	4.62	6.80	17	"	"	"
90			$20\times\frac{1}{2}$	"	24.64	2.53	520.5	1126.3	4.59	6.75	"	"	"	"
91	12	30.00	$16\times\frac{3}{8}$	9	23.58	1.57	494.9	611.4	4.58	5.08	13	2	.51	$\frac{7}{8}$
92			$16\times\frac{7}{16}$	"	24.58	1.77	517.3	632.7	4.58	5.06	"	"	"	"
93			$18\times\frac{3}{8}$	11	24.33	1.71	510.1	865.7	4.57	5.96	15	"	"	"
94			$18\times\frac{7}{16}$	"	25.46	1.92	534.1	896.0	4.58	5.93	"	"	"	"
95			$20\times\frac{7}{16}$	13	26.33	2.06	549.8	1211.1	4.56	6.78	17	"	"	"
96			$20\times\frac{1}{2}$	"	27.58	2.34	567.6	1252.7	4.53	6.73	"	"	"	"
97	15	33.9	$18\times\frac{3}{8}$	$10\frac{5}{8}$	26.55	1.96	922.8	936.7	5.90	5.94	15	$2\frac{3}{16}$	.40	$\frac{7}{8}$
98			$18\times\frac{7}{16}$	"	27.68	2.20	961.0	967.0	5.89	5.91	"	"	"	"
99			$20\times\frac{7}{16}$	$12\frac{5}{8}$	28.55	2.36	986.7	1307.1	5.88	6.76	17	"	"	"
100			$20\times\frac{1}{2}$	"	29.80	2.60	1024.5	1348.7	5.86	6.72	"	"	"	"
101			$22\times\frac{1}{2}$	$14\frac{5}{8}$	30.80	2.77	1050.2	1761.1	5.84	7.56	19	"	"	"
102			$22\times\frac{9}{16}$	"	32.18	3.00	1085.5	1816.5	5.81	7.50	"	"	"	"
103	15	35.00	$18\times\frac{3}{8}$	$10\frac{5}{8}$	27.21	1.90	940.5	965.7	5.87	5.95	15	$2\frac{3}{16}$	.43	$\frac{7}{8}$
104			$18\times\frac{7}{16}$	"	28.34	2.14	979.7	996.0	5.87	5.92	"	"	"	"
105			$20\times\frac{7}{16}$	$12\frac{5}{8}$	29.21	2.30	1005.6	1346.7	5.86	6.78	17	"	"	"
106			$20\times\frac{1}{2}$	"	30.46	2.53	1044.4	1388.3	5.84	6.74	"	"	"	"
107			$22\times\frac{1}{2}$	$14\frac{5}{8}$	31.46	2.70	1070.8	1811.7	5.82	7.58	19	"	"	"
108			$22\times\frac{9}{16}$	"	32.84	2.92	1107.9	1867.1	5.79	7.52	"	"	"	"
109	15	40.00	$18\times\frac{3}{8}$	$10\frac{3}{8}$	30.15	1.71	1005.1	1039.3	5.76	5.86	15	$2\frac{5}{16}$	.52	$\frac{7}{8}$
110			$18\times\frac{7}{16}$	"	31.28	1.94	1047.0	1069.6	5.77	5.84	"	"	"	"
111			$20\times\frac{7}{16}$	$12\frac{3}{8}$	32.15	2.09	1074.8	1453.5	5.77	6.71	17	"	"	"
112			$20\times\frac{1}{2}$	"	33.40	2.31	1116.7	1495.1	5.77	6.68	"	"	"	"
113			$22\times\frac{1}{2}$	$14\frac{3}{8}$	34.40	2.47	1145.4	1956.5	5.76	7.52	19	"	"	"
114			$22\times\frac{9}{16}$	"	35.78	2.68	1186.2	2011.9	5.75	7.48	"	"	"	"
115	15	45.00	$18\times\frac{3}{8}$	$10\frac{1}{4}$	33.09	1.56	1068.2	1127.9	5.67	5.82	15	$2\frac{3}{8}$	.62	$\frac{7}{8}$
116			$18\times\frac{7}{16}$	"	34.22	1.77	1112.0	1158.2	5.69	5.81	"	"	"	"
117			$20\times\frac{7}{16}$	$12\frac{1}{4}$	35.09	1.92	1141.9	1577.3	5.69	6.69	17	"	"	"
118			$20\times\frac{1}{2}$	"	36.34	2.12	1186.4	1618.9	5.70	6.66	"	"	"	"
119			$22\times\frac{1}{2}$	$14\frac{1}{4}$	37.34	2.28	1217.2	2120.7	5.70	7.52	19	"	"	"
120			$22\times\frac{9}{16}$	"	38.72	2.48	1260.6	2176.1	5.70	7.48	"	"	"	"

TABLE 83.

Properties of Top Chord Sections.

Properties of Highway Bridge Top Chord Sections.



Four Angles and Three Plates.

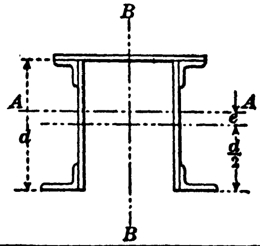
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches^2.	Inches.	Inches^4.	Inches^4.	Inches.	Inches.
				12" × 14" Section. A Series.						
*1	12"x¼"	14"x5/16"	2½x2½x5/16	2½x2½x5/16	16.26	1.66	359	351	4.70	4.65
2	" 5/16	"	"	"	17.76	1.52	381	378	4.63	4.61
3	" ⅜	"	"	"	19.26	1.40	402	404	4.57	4.58
4	" 7/16	"	"	"	20.76	1.30	423	429	4.52	4.55
5	" ½	"	"	"	22.26	1.21	443	453	4.46	4.52
6	" 9/16	"	"	"	23.76	1.14	463	476	4.41	4.48
7	" ⅝	"	"	"	25.26	1.07	483	498	4.37	4.44
*8	12x¼	14x5/16	2½x2½x5/16	2½x2½x⅜	16.80	1.45	384	367	4.78	4.67
9	" 5/16	"	"	"	18.30	1.33	405	394	4.70	4.63
10	" ⅜	"	"	"	19.80	1.23	425	420	4.63	4.60
11	" 7/16	"	"	"	21.30	1.14	445	445	4.57	4.57
12	" ½	"	"	"	22.80	1.07	465	469	4.52	4.54
13	" 9/16	"	"	"	24.30	1.00	485	492	4.47	4.50
14	" ⅝	"	"	"	25.80	0.94	504	514	4.42	4.47
*15	12x¼	14x5/16	2½x2½x5/16	2½x2½x7/16	17.32	1.25	405	383	4.83	4.70
16	" 5/16	"	"	"	18.82	1.16	425	410	4.75	4.66
17	" ⅜	"	"	"	20.32	1.06	445	436	4.68	4.63
18	" 7/16	"	"	"	21.82	0.99	465	461	4.61	4.59
19	" ½	"	"	"	23.32	0.93	484	485	4.55	4.56
20	" 9/16	"	"	"	24.82	0.87	503	508	4.50	4.52
21	" ⅝	"	"	"	26.32	0.82	522	530	4.46	4.49
*22	12x¼	14x5/16	2½x2½x5/16	2½x2½x½	17 82	1.07	425	398	4.88	4.73
23	" 5/16	"	"	"	19.32	0.99	444	425	4 79	4 69
24	" ⅜	"	"	"	20.82	0.92	463	451	4.71	4.65
25	" 7/16	"	"	"	22.32	0.86	483	476	4.65	4.62
26	" ½	"	"	"	23.82	0.80	502	500	4.59	4.58
27	" 9/16	"	"	"	25.32	0.75	521	523	4.54	4.55
28	" ⅝	"	"	"	26.82	0.71	540	545	4.49	4.51
*29	12x¼	14x5/16	2½x2½x5/16	2½x2½x9/16	18.32	0.91	442	414	4.91	4.75
30	" 5/16	"	"	"	19.82	0.84	461	441	4.82	4.71
31	" ⅜	"	"	"	21 32	0.78	480	467	4.74	4 68
32	" 7/16	"	"	"	22.82	0.73	499	492	4.67	4.64
33	" ½	"	"	"	24.32	0.68	518	516	4.61	4.60
34	" 9/16	"	"	"	25.82	0.64	536	539	4.56	4.56
35	" ⅝	"	"	"	27.32	0.61	555	561	4.51	4.53

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 83.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Highway Bridge Top Chord Sections.



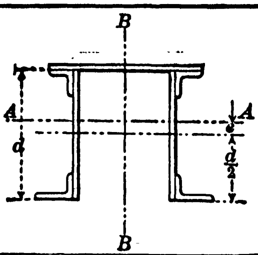
Four Angles and Three Plates.

Section Number.	Plates.		Angles.		Gross Area.	Eccentricity.	Moments of Inertia		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.			Axis A–A.	Axis B–B.	Axis A–A.	Axis B–B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
				12″ × 14″ Section. B Series.						
*36	12x¼	14x5/16	2½x2½x5/16	3x2½x5/16	16.58	1.52	377	368	4.77	4.71
37	" 5/16	"	"	"	18.08	1.39	398	395	4.69	4.67
38	" ⅜	"	"	"	19.58	1.28	419	421	4.62	4.64
39	" 7/16	"	"	"	21.08	1.19	439	446	4.56	4.60
40	" ½	"	"	"	22.58	1.11	459	470	4.51	4.56
41	" 9/16	"	"	"	24.08	1.04	479	493	4.46	4.52
42	" ⅝	"	"	"	25.58	0.98	498	515	4.41	4.49
*43	12x¼	14x5/16	2½x2½x5/16	3x2½x⅜	17.18	1.29	403	387	4.84	4.74
44	" 5/16	"	"	"	18.68	1.18	423	414	4.76	4.70
45	" ⅜	"	"	"	20.18	1.09	443	440	4.69	4.67
46	" 7/16	"	"	"	21.68	1.02	463	465	4.62	4.63
47	" ½	"	"	"	23.18	0.95	482	489	4.56	4.59
48	" 9/16	"	"	"	24.68	0.90	501	512	4.51	4.55
49	" ⅝	"	"	"	26.18	0.85	520	534	4.46	4.51
*50	12x¼	14x5/16	2½x2½x5/16	3x2½x7/16	17.76	1.07	427	406	4.90	4.78
51	" 5/16	"	"	"	19.26	0.99	446	433	4.81	4.74
52	" ⅜	"	"	"	20.76	0.92	465	459	4.73	4.70
53	" 7/16	"	"	"	22.26	0.86	485	484	4.67	4.66
54	" ½	"	"	"	23.76	0.80	504	508	4.60	4.62
55	" 9/16	"	"	"	25.26	0.75	523	531	4.55	4.58
56	" ⅝	"	"	"	26.76	0.71	541	553	4.50	4.54
*57	12x¼	14x5/16	2½x2½x5/16	3x2½x½	18.32	0.88	447	424	4.94	4.81
58	" 5/16	"	"	"	19.82	0.82	466	451	4.85	4.77
59	" ⅜	"	"	"	21.32	0.76	485	477	4.77	4.73
60	" 7/16	"	"	"	22.82	0.71	504	502	4.70	4.69
61	" ½	"	"	"	24.32	0.67	522	526	4.63	4.65
62	" 9/16	"	"	"	25.82	0.63	541	549	4.57	4.61
63	" ⅝	"	"	"	27.32	0.59	560	571	4.52	4.57
*64	12x¼	14x5/16	2½x2½x5/16	3x2½x9/16	18.88	0.71	466	443	4.97	4.84
65	" 5/16	"	"	"	20.38	0.66	485	470	4.88	4.80
66	" ⅜	"	"	"	21.88	0.61	504	496	4.80	4.76
67	" 7/16	"	"	"	23.38	0.57	522	521	4.73	4.72
68	" ½	"	"	"	24.88	0.54	541	545	4.66	4.68
69	" 9/16	"	"	"	26.38	0.51	559	568	4.60	4.64
70	" ⅝	"	"	"	27.88	0.48	578	590	4.55	4.60

* Spacing of rivet lines of web greater than 30 × thickness of plate.

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Highway Bridge Top Chord Sections.



Four Angles and Three Plates.

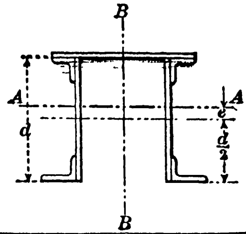
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
					14" × 16" Section. A Series.					
*71	14x1/4	16x3/8	3x3x5/16	3x3x5/16	20.12	2.14	606	546	5.49	5.21
*72	“ 5/16	“	“	“	21.87	1.97	641	585	5.41	5.17
73	“ 3/8	“	“	“	23.62	1.82	677	623	5.35	5.13
74	“ 7/16	“	“	“	25.37	1.70	711	660	5.29	5.10
75	“ 1/2	“	“	“	27.12	1.59	744	696	5.24	5.06
76	“ 9/16	“	“	“	28.87	1.49	777	731	5.19	5.02
77	“ 5/8	“	“	“	30.62	1.41	808	765	5.14	4.99
*78	14x1/4	16x3/8	3x3x5/16	3x3x3/8	20.78	1.88	648	570	5.58	5.24
*79	“ 5/16	“	“	“	22.53	1.73	683	609	5.50	5.20
80	“ 3/8	“	“	“	24.28	1.61	716	647	5.43	5.16
81	“ 7/16	“	“	“	26.03	1.50	749	684	5.36	5.12
82	“ 1/2	“	“	“	27.78	1.41	781	720	5.30	5.09
83	“ 9/16	“	“	“	29.53	1.32	813	755	5.25	5.06
84	“ 5/8	“	“	“	31.28	1.25	845	789	5.20	5.04
*85	14x1/4	16x3/8	3x3x5/16	3x3x7/16	21.44	1.64	688	594	5.66	5.26
*86	“ 5/16	“	“	“	23.19	1.52	722	633	5.58	5.22
87	“ 3/8	“	“	“	24.94	1.41	754	671	5.50	5.18
88	“ 7/16	“	“	“	26.69	1.32	786	708	5.42	5.15
89	“ 1/2	“	“	“	28.44	1.24	816	744	5.36	5.11
90	“ 9/16	“	“	“	30.19	1.17	848	779	5.30	5.08
91	“ 5/8	“	“	“	31.94	1.10	879	813	5.24	5.04
*92	14x1/4	16x3/8	3x3x5/16	3x3x1/2	22.06	1.43	721	618	5.72	5.29
*93	“ 5/16	“	“	“	23.81	1.32	755	657	5.63	5.25
94	“ 3/8	“	“	“	25.56	1.23	786	695	5.54	5.21
95	“ 7/16	“	“	“	27.31	1.15	818	732	5.47	5.18
96	“ 1/2	“	“	“	29.06	1.08	848	768	5.40	5.14
97	“ 9/16	“	“	“	30.81	1.02	879	803	5.34	5.10
98	“ 5/8	“	“	“	32.56	0.97	909	837	5.28	5.07
*99	14x1/4	16x3/8	3x3x5/16	3x3x9/16	22.68	1.23	756	641	5.77	5.31
*100	“ 5/16	“	“	“	24.43	1.14	787	680	5.67	5.27
101	“ 3/8	“	“	“	26.18	1.07	817	718	5.58	5.24
102	“ 7/16	“	“	“	27.93	1.00	848	755	5.50	5.20
103	“ 1/2	“	“	“	29.68	0.94	878	791	5.43	5.16
104	“ 9/16	“	“	“	31.43	0.89	908	826	5.37	5.12
105	“ 5/8	“	“	“	33.18	0.84	938	860	5.32	5.09

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 83.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Highway Bridge Top Chord Sections.



Four Angles and Three Plates.

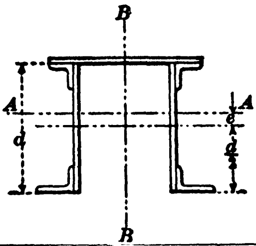
Section Number.	Plates. Web	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches
*106	14x$\frac{1}{4}$	16x$\frac{3}{8}$	3x3x$\frac{5}{16}$	3x3x$\frac{3}{8}$	23.28	1.05	784	665	5.80	5.34
*107	" $\frac{5}{16}$	"	"	"	25.03	0.98	814	704	5.70	5.30
108	" $\frac{3}{8}$	"	"	"	26.78	0.92	844	742	5.61	5.26
109	" $\frac{7}{16}$	"	"	"	28.53	0.86	875	779	5.53	5.22
110	" $\frac{1}{2}$	"	"	"	30.28	0.81	904	815	5.46	5.19
111	" $\frac{9}{16}$	"	"	"	32.03	0.76	934	850	5.39	5.15
112	" $\frac{5}{8}$	"	"	"	33.78	0.73	963	884	5.34	5.12
14″ × 16″ Section. B Series.										
*113	14x$\frac{1}{4}$	16x$\frac{3}{8}$	3x3x$\frac{5}{16}$	4x3x$\frac{5}{16}$	20.74	1.87	654	590	5.62	5.33
*114	" $\frac{5}{16}$	"	"	"	22.49	1.72	689	629	5.53	5.29
115	" $\frac{3}{8}$	"	"	"	24.24	1.60	722	667	5.46	5.24
116	" $\frac{7}{16}$	"	"	"	25.99	1.49	755	704	5.39	5.20
117	" $\frac{1}{2}$	"	"	"	27.74	1.40	788	740	5.33	5.16
118	" $\frac{9}{16}$	"	"	"	29.49	1.32	819	775	5 27	5 12
119	" $\frac{5}{8}$	"	"	"	31.24	1.24	851	809	5.22	5.08
*120	14x$\frac{1}{4}$	16x$\frac{3}{8}$	3x3x$\frac{5}{16}$	4x3x$\frac{3}{8}$	21.52	1.57	704	624	5.72	5.38
*121	" $\frac{5}{16}$	"	"	"	23.27	1.46	736	663	5.62	5.34
122	" $\frac{3}{8}$	"	"	"	25.02	1 36	768	701	5.54	5.29
123	" $\frac{7}{16}$	"	"	"	26.77	1 27	800	738	5.46	5.25
124	" $\frac{1}{2}$	"	"	"	28.52	1 19	831	774	5.40	5.21
125	" $\frac{9}{16}$	"	"	"	30.27	1 12	862	809	5 34	5.17
126	" $\frac{5}{8}$	"	"	"	32.02	1.06	892	843	5.28	5.13
*127	14x$\frac{1}{4}$	16x$\frac{3}{8}$	3x3x$\frac{5}{16}$	4x3x$\frac{7}{16}$	22.30	1.31	748	658	5.79	5.43
*128	" $\frac{5}{16}$	"	"	"	24.05	1.21	780	697	5.69	5.38
129	" $\frac{3}{8}$	"	"	"	25.80	1.13	810	735	5.60	5.33
130	" $\frac{7}{16}$	"	"	"	27.55	1.06	841	772	5.52	5 29
131	" $\frac{1}{2}$	"	"	"	29.30	1.00	872	808	5.45	5.25
132	" $\frac{9}{16}$	"	"	"	31.05	0.94	902	843	5 38	5.21
133	" $\frac{5}{8}$	"	"	"	32.80	0 89	932	877	5.33	5.17
*134	14x$\frac{1}{4}$	16x$\frac{3}{8}$	3x3x$\frac{5}{16}$	4x3x$\frac{1}{2}$	23.06	1.08	787	690	5.84	5.47
*135	" $\frac{5}{16}$	"	"	"	24.81	1.00	817.	729	5.73	5.42
136	" $\frac{3}{8}$	"	"	"	26.56	0.93	848	767	5.65	5.37
137	" $\frac{7}{16}$	"	"	"	28 31	0.88	877	804	5 56	5.32
138	" $\frac{1}{2}$	"	"	"	30.06	0.83	907	840	5.49	5.28
139	" $\frac{9}{16}$	"	"	"	31.81	0.78	938	875	5.42	5.24
140	" $\frac{5}{8}$	"	"	"	33 56	0.74	967	909	5.37	5.20

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 83.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Highway Bridge Top Chord Sections



Four Angles and Three Plates

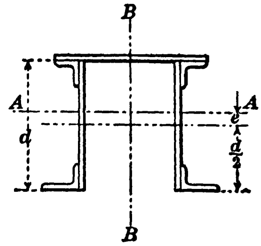
Section Number.	Plates.		Angles.		Gross Area.	Eccen-tricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.	A	e	Axis A-A. I_A	Axis B-B. I_B	Axis A-A. r_A	Axis B-B. r_B
	Inches.	Inches	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*141	14x1/4	16x3/8	3x3x5/16	4x3x9/16	23.80	0.85	824	724	5.88	5.51
*142	“ 5/16	“	“	“	25.55	0.79	853	763	5.77	5.47
143	“ 3/8	“	“	“	27.30	0.74	883	801	5.68	5.42
144	“ 7/16	“	“	“	29.05	0.69	913	838	5.60	5.37
145	“ 1/2	“	“	“	30.80	0.65	942	874	5.52	5.32
146	“ 9/16	“	“	“	32.55	0.62	971	909	5.46	5.28
147	“ 5/8	“	“	“	34.30	0.59	1000	943	5.40	5.24
*148	14x1/4	16x3/8	3x3x5/16	4x3x5/8	24.52	0.65	856	756	5.91	5.55
*149	“ 5/16	“	“	“	26.27	0.61	884	795	5.80	5.50
150	“ 3/8	“	“	“	28.02	0.57	914	833	5.71	5.45
151	“ 7/16	“	“	“	29.77	0.54	942	870	5.62	5.41
152	“ 1/2	“	“	“	31.52	0.51	972	906	5.55	5.36
153	“ 9/16	“	“	“	33.27	0.48	1001	941	5.48	5.32
154	“ 5/8	“	“	“	35.02	0.46	1030	975	5.42	5.28
14″ × 17″ Section.										
*155	14x1/4	17x3/8	3x3x5/16	4x3x5/16	21.12	1.96	665	704	5.61	5.77
*156	“ 5/16	“	“	“	22.87	1.82	699	751	5.52	5.73
157	“ 3/8	“	“	“	24.62	1.69	734	797	5.45	5.68
158	“ 7/16	“	“	“	26.37	1.57	767	842	5.39	5.65
159	“ 1/2	“	“	“	28.12	1.47	800	886	5.33	5.61
160	“ 9/16	“	“	“	29.87	1.39	833	929	5.28	5.57
161	“ 5/8	“	“	“	31.62	1.31	864	971	5.22	5.54
*162	14x1/4	17x3/8	3x3x5/16	4x3x3/8	21.90	1.67	715	743	5.71	5.82
*163	“ 5/16	“	“	“	23.65	1.55	748	790	5.62	5.77
164	“ 3/8	“	“	“	25.40	1.44	780	836	5.54	5.73
165	“ 7/16	“	“	“	27.15	1.35	813	881	5.47	5.69
166	“ 1/2	“	“	“	28.90	1.27	845	925	5.41	5.65
167	“ 9/16	“	“	“	30.65	1.19	875	968	5.35	5.62
168	“ 5/8	“	“	“	32.40	1.13	907	1010	5.29	5.58
*169	14x1/4	17x3/8	3x3x5/16	4x3x7/16	22.68	1.40	761	781	5.79	5.86
*170	“ 5/16	“	“	“	24.43	1.30	792	828	5.69	5.82
171	“ 3/8	“	“	“	26.18	1.22	824	874	5.60	5.77
172	“ 7/16	“	“	“	27.93	1.14	855	919	5.53	5.73
173	“ 1/2	“	“	“	29.68	1.07	886	963	5.46	5.69
174	“ 9/16	“	“	“	31.43	1.01	917	1006	5.40	5.65
175	“ 5/8	“	“	“	33.18	0.96	946	1048	5.34	5.61

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 83.—*Continued.*

Properties of Top Chord Sections.

Properties of Highway Bridge Top Chord Sections.



Four Angles and Three Plates.

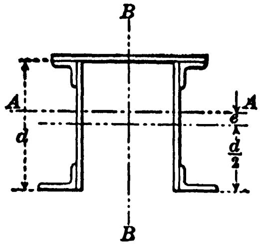
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*176	14x$\frac{1}{4}$	17x$\frac{3}{8}$	3x3x$\frac{5}{16}$	4x3x$\frac{1}{2}$	23.44	1.17	801	819	5.84	5.90
*177	" $\frac{5}{16}$	"	"	"	25.19	1.09	832	866	5.75	5.86
178	" $\frac{3}{8}$	"	"	"	26.94	1.02	862	912	5.66	5.82
179	" $\frac{7}{16}$	"	"	"	28.69	0.96	893	957	5.58	5.78
180	" $\frac{1}{2}$	"	"	"	30.44	0.90	923	1001	5.51	5.74
181	" $\frac{9}{16}$	"	"	"	32.19	0.85	953	1044	5.44	5.70
182	" $\frac{5}{8}$	"	"	"	33.94	0.81	983	1086	5.38	5.66
*183	14x$\frac{1}{4}$	17x$\frac{3}{8}$	3x3x$\frac{5}{16}$	4x3x$\frac{9}{16}$	24.18	0.94	839	858	5.89	5.95
*184	" $\frac{5}{16}$	"	"	"	25.93	0.88	869	905	5.79	5.90
185	" $\frac{3}{8}$	"	"	"	27.68	0.82	898	951	5.69	5.86
186	" $\frac{7}{16}$	"	"	"	29.43	0.77	928	996	5.61	5.81
187	" $\frac{1}{2}$	"	"	"	31.18	0.73	958	1040	5.54	5.77
188	" $\frac{9}{16}$	"	"	"	32.93	0.69	987	1083	5.47	5.73
189	" $\frac{5}{8}$	"	"	"	34.68	0.66	1017	1125	5.41	5.69
190	" $\frac{11}{16}$	"	"	"	36.43	0.63	1046	1166	5.35	5.65
*191	14x$\frac{1}{4}$	17x$\frac{3}{8}$	3x3x$\frac{5}{16}$	4x3x$\frac{5}{8}$	24.90	0.75	871	895	5.91	5.99
*192	" $\frac{5}{16}$	"	"	"	26.65	0.70	901	942	5.81	5.94
193	" $\frac{3}{8}$	"	"	"	28.40	0.66	930	988	5.72	5.89
194	" $\frac{7}{16}$	"	"	"	30.15	0.62	959	1033	5.64	5.85
195	" $\frac{1}{2}$	"	"	"	31.90	0.59	988	1077	5.56	5.81
196	" $\frac{9}{16}$	"	"	"	33.65	0.56	1018	1120	5.50	5.77
197	" $\frac{5}{8}$	"	"	"	35.40	0.53	1047	1162	5.44	5.73
198	" $\frac{11}{16}$	"	"	"	37.15	0.50	1076	1203	5.38	5.69
199	" $\frac{3}{4}$	"	"	"	38.90	0.48	1105	1243	5.33	5.65
*200	14x$\frac{1}{4}$	17x$\frac{3}{8}$	3x3x$\frac{5}{16}$	4x3x$\frac{11}{16}$	25.62	0.57	903	931	5.94	6.03
*201	" $\frac{5}{16}$	"	"	"	27.37	0.53	931	978	5.84	5.98
202	" $\frac{3}{8}$	"	"	"	29.12	0.50	961	1024	5.75	5.93
203	" $\frac{7}{16}$	"	"	"	30.87	0.47	990	1069	5.66	5.88
204	" $\frac{1}{2}$	"	"	"	32.62	0.45	1018	1113	5.59	5.84
205	" $\frac{9}{16}$	"	"	"	34.37	0.42	1048	1156	5.53	5.80
206	" $\frac{5}{8}$	"	"	"	36.12	0.40	1076	1198	5.46	5.76
207	" $\frac{11}{16}$	"	"	"	37.87	0.38	1105	1239	5.40	5.72
208	" $\frac{3}{4}$	"	"	"	39.62	0.37	1135	1279	5.35	5.68

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 83.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Highway Bridge Top Chord Sections.



Four Angles and Three Plates.

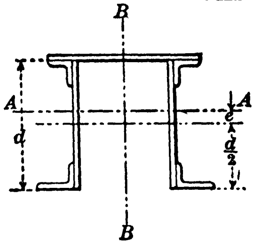
Section Number.	Plates.		Angles.		Gross Area	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.			Axis A–A.	Axis B–B.	Axis A–A.	Axis B–B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
				15″ × 17″ Section.						
*209	15x5/16	17x3/8	3x3x5/16	4x3x5/16	23.50	1.89	821	766	5.91	5.71
*210	" 3/8	"	"	"	25.38	1.75	862	816	5.83	5.67
211	" 7/16	"	"	"	27.25	1.63	902	865	5.75	5.63
212	" 1/2	"	"	"	29.13	1.52	942	912	5.68	5.59
213	" 9/16	"	"	"	31.00	1.43	983	958	5.62	5.56
214	" 5/8	"	"	"	32.88	1.35	1021	1003	5.57	5.52
215	" 11/16	"	"	"	34.75	1.28	1059	1047	5.52	5.49
216	" 3/4	"	"	"	36.63	1.21	1097	1090	5.47	5.46
*217	15x5/16	17x3/8	3x3x5/16	4x3x3/8	24.28	1.61	877	807	6.01	5.76
*218	" 3/8	"	"	"	26.16	1.49	917	857	5.92	5.72
219	" 7/16	"	"	"	28.03	1.39	956	906	5.84	5.68
220	" 1/2	"	"	"	29.91	1.31	994	953	5.76	5.64
221	" 9/16	"	"	"	31.78	1.23	1033	999	5.70	5.60
222	" 5/8	"	"	"	33.66	1.16	1071	1044	5.64	5.57
223	" 11/16	"	"	"	35.53	1.10	1108	1088	5.58	5.54
224	" 3/4	"	"	"	37.41	1.05	1145	1131	5.53	5.50
*225	15x5/16	17x3/8	3x3x5/16	4x3x7/16	25.06	1.36	929	845	6.08	5.81
*226	" 3/8	"	"	"	26.94	1.26	967	895	5.98	5.76
226	" 7/16	"	"	"	28.81	1.18	1005	944	5.90	5.72
227	" 1/2	"	"	"	30.69	1.11	1042	991	5.82	5.68
228	" 9/16	"	"	"	32.56	1.04	1080	1037	5.76	5.64
229	" 5/8	"	"	"	34.44	0.99	1117	1082	5.69	5.61
230	" 11/16	"	"	"	36.31	0.94	1154	1126	5.63	5.57
231	" 3/4	"	"	"	38.19	0.89	1191	1169	5.58	5.53
*232	15x5/16	17x3/8	3x3x5/16	4x3x1/2	25.82	1.13	973	883	6.14	5.84
*233	" 3/8	"	"	"	27.70	1.05	1010	933	6.04	5.80
234	" 7/16	"	"	"	29.57	0.99	1047	982	5.95	5.76
235	" 1/2	"	"	"	31.45	0.93	1084	1029	5.87	5.72
236	" 9/16	"	"	"	33.32	0.88	1121	1075	5.79	5.68
237	" 5/8	"	"	"	35.20	0.83	1158	1120	5.73	5.64
238	" 11/16	"	"	"	37.07	0.79	1194	1164	5.68	5.61
239	" 3/4	"	"	"	38.95	0.75	1230	1207	5.62	5.57

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 83.—*Continued.*

Properties of Top Chord Sections.

Properties of
Highway Bridge
Top Chord Sections.



Four Angles and Three Plates.

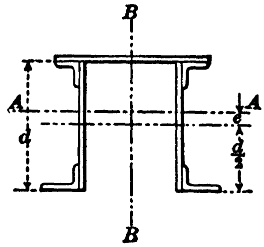
Section Number.	Plates.		Angles.		Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.			Axis A–A.	Axis B–B.	Axis A–A.	Axis B–B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*240	15x$\frac{5}{16}$	17x$\frac{3}{8}$	3x3x$\frac{5}{16}$	4x3x$\frac{9}{16}$	26.56	0.91	1016	920	6.18	5.88
*241	" $\frac{3}{8}$	"	"	"	28.44	0.85	1052	970	6.08	5.84
242	" $\frac{7}{16}$	"	"	"	30.31	0.80	1089	1019	5.99	5.80
243	" $\frac{1}{2}$	"	"	"	32.19	0.75	1125	1066	5.91	5.76
244	" $\frac{9}{16}$	"	"	"	34.06	0.71	1161	1112	5.84	5.72
245	" $\frac{5}{8}$	"	"	"	35.94	0.68	1197	1157	5.77	5.68
246	" $\frac{11}{16}$	"	"	"	37.81	0.64	1233	1201	5.71	5.64
247	" $\frac{3}{4}$	"	"	"	39.69	0.61	1269	1244	5.65	5.60
*248	15x$\frac{5}{16}$	17x$\frac{3}{8}$	3x3x$\frac{5}{16}$	4x3x$\frac{5}{8}$	27.28	0.72	1055	959	6.22	5.92
*249	" $\frac{3}{8}$	"	"	"	29.16	0.67	1091	1009	6.12	5.88
250	" $\frac{7}{16}$	"	"	"	31.03	0.63	1127	1058	6.03	5.84
251	" $\frac{1}{2}$	"	"	"	32.91	0.60	1162	1105	5.94	5.80
252	" $\frac{9}{16}$	"	"	"	34.78	0.57	1199	1151	5.87	5.75
253	" $\frac{5}{8}$	"	"	"	36.66	0.54	1234	1196	5.80	5.71
254	" $\frac{11}{16}$	"	"	"	38.53	0.51	1270	1240	5.74	5.67
255	" $\frac{3}{4}$	"	"	"	40.41	0.49	1305	1283	5.68	5.63
*256	15x$\frac{5}{16}$	17x$\frac{3}{8}$	3x3x$\frac{5}{16}$	4x3x$\frac{11}{16}$	28.00	0.54	1089	995	6.24	5.96
*257	" $\frac{3}{8}$	"	"	"	29.88	0.51	1124	1045	6.14	5.91
258	" $\frac{7}{16}$	"	"	"	31.75	0.48	1160	1094	6.04	5.87
259	" $\frac{1}{2}$	"	"	"	33.63	0.45	1195	1141	5.96	5.82
260	" $\frac{9}{16}$	"	"	"	35.50	0.43	1231	1187	5.89	5.78
261	" $\frac{5}{8}$	"	"	"	37.38	0.41	1267	1232	5.82	5.74
262	" $\frac{11}{16}$	"	"	"	39.25	0.39	1302	1276	5.76	5.70
263	" $\frac{3}{4}$	"	"	"	41.13	0.37	1337	1319	5.70	5.66
15″ × 18″ Section.										
*264	15x$\frac{5}{16}$	18x$\frac{7}{16}$	3x3x$\frac{5}{16}$	4x3x$\frac{5}{16}$	25.00	2.25	872	931	5.90	6.10
*265	" $\frac{3}{8}$	"	"	"	26.88	2.09	915	991	5.83	6.07
266	" $\frac{7}{16}$	"	"	"	28.75	1.95	958	1050	5.77	6.04
267	" $\frac{1}{2}$	"	"	"	30.63	1.83	1000	1108	5.71	6.01
268	" $\frac{9}{16}$	"	"	"	32.50	1.73	1042	1164	5.66	5.98
269	" $\frac{5}{8}$	"	"	"	34.38	1.64	1082	1219	5.61	5.95
270	" $\frac{11}{16}$	"	"	"	36.25	1.55	1122	1272	5.56	5.92
271	" $\frac{3}{4}$	"	"	"	38.13	1.47	1161	1324	5.52	5.89

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 83.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Highway Bridge Top Chord Sections.



Four Angles and Three Plates.

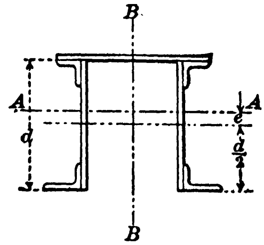
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*272	15x$\frac{5}{16}$	18x$\frac{7}{16}$	3x3x$\frac{5}{16}$	4x3x$\frac{3}{8}$	25.78	1.97	933	976	6.01	6.15
*273	“ $\frac{3}{8}$	“	“	“	27.66	1.84	974	1036	5.93	6.12
274	“ $\frac{7}{16}$	“	“	“	29.53	1.72	1015	1095	5.86	6.09
275	“ $\frac{1}{2}$	“	“	“	31.41	1.62	1055	1153	5.79	6.06
276	“ $\frac{9}{16}$	“	“	“	33.28	1.53	1096	1209	5.73	6.02
277	“ $\frac{5}{8}$	“	“	“	35.16	1.45	1135	1264	5.68	5.99
278	“ $\frac{11}{16}$	“	“	“	37.03	1.37	1174	1317	5.63	5.96
279	“ $\frac{3}{4}$	“	“	“	38.91	1.31	1212	1369	5.58	5.93
*280	15x$\frac{5}{16}$	18x$\frac{7}{16}$	3x3x$\frac{5}{16}$	4x3x$\frac{7}{16}$	26.56	1.72	988	1020	6.10	6.20
*281	“ $\frac{3}{8}$	“	“	“	28.44	1.61	1028	1080	6.01	6.16
282	“ $\frac{7}{16}$	“	“	“	30.31	1.51	1068	1139	5.93	6.13
283	“ $\frac{1}{2}$	“	“	“	32.19	1.42	1107	1197	5.86	6.09
284	“ $\frac{9}{16}$	“	“	“	34.06	1.35	1146	1253	5.79	6.06
285	“ $\frac{5}{8}$	“	“	“	35.94	1.28	1184	1308	5.74	6.03
286	“ $\frac{11}{16}$	“	“	“	37.81	1.21	1222	1361	5.68	6.00
287	“ $\frac{3}{4}$	“	“	“	39.69	1.15	1260	1413	5.63	5.97
*288	15x$\frac{5}{16}$	18x$\frac{7}{16}$	3x3x$\frac{5}{16}$	4x3x$\frac{1}{2}$	27.32	1.50	1038	1063	6.16	6.24
*289	“ $\frac{3}{8}$	“	“	“	29.20	1.40	1077	1123	6.07	6.20
290	“ $\frac{7}{16}$	“	“	“	31.07	1.32	1115	1182	5.99	6.17
291	“ $\frac{1}{2}$	“	“	“	32.95	1.24	1153	1240	5.92	6.14
292	“ $\frac{9}{16}$	“	“	“	34.82	1.18	1192	1296	5.85	6.10
293	“ $\frac{5}{8}$	“	“	“	36.70	1.12	1229	1351	5.79	6.07
294	“ $\frac{11}{16}$	“	“	“	38.57	1.06	1266	1404	5.73	6.04
295	“ $\frac{3}{4}$	“	“	“	40.45	1.01	1303	1456	5.68	6.00
*296	15x$\frac{5}{16}$	18x$\frac{7}{16}$	3x3x$\frac{5}{16}$	4x3x$\frac{9}{16}$	28.06	1.28	1085	1107	6.21	6.28
*297	“ $\frac{3}{8}$	“	“	“	29.94	1.20	1123	1167	6.12	6.24
298	“ $\frac{7}{16}$	“	“	“	31.81	1.13	1160	1226	6.04	6.20
299	“ $\frac{1}{2}$	“	“	“	33.69	1.07	1197	1284	5.96	6.17
300	“ $\frac{9}{16}$	“	“	“	35.56	1.01	1235	1340	5.89	6.14
301	“ $\frac{5}{8}$	“	“	“	37.44	0.96	1272	1395	5.83	6.10
302	“ $\frac{11}{16}$	“	“	“	39.31	0.92	1309	1448	5.77	6.06
303	“ $\frac{3}{4}$	“	“	“	41.19	0.88	1345	1500	5.71	6.03

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 83.—*Continued.*

Properties of Top Chord Sections.

Properties of Highway Bridge Top Chord Sections.



Four Angles and Three Plates.

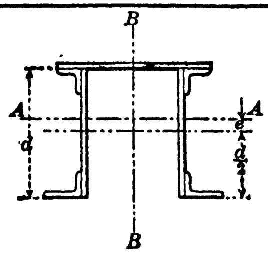
Section Number.	Plates.		Angles.		Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.			Axis A–A.	Axis B–B.	Axis A–A.	Axis B–B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*304	15x$\frac{5}{16}$	18x$\frac{7}{16}$	3x3x$\frac{5}{16}$	4x3x$\frac{5}{8}$	28.78	1.09	1127	1149	6.26	6.31
*305	" $\frac{3}{8}$	"	"	"	30.66	1.03	1164	1209	6.16	6.27
306	" $\frac{7}{16}$	"	"	"	32.53	0.97	1201	1268	6.07	6.24
307	" $\frac{1}{2}$	"	"	"	34.41	0.92	1237	1326	5.99	6.20
308	" $\frac{9}{16}$	"	"	"	36.28	0.87	1275	1382	5.92	6.17
309	" $\frac{5}{8}$	"	"	"	38.16	0.83	1311	1437	5.86	6.14
310	" $\frac{11}{16}$	"	"	"	40.03	0.79	1347	1490	5.80	6.10
311	" $\frac{3}{4}$	"	"	"	41.91	0.75	1383	1542	5.74	6.06
*312	15x$\frac{5}{16}$	18x$\frac{7}{16}$	3x3x$\frac{5}{16}$	4x3x$\frac{11}{16}$	29.50	0.92	1165	1191	6.28	6.36
*313	" $\frac{3}{8}$	"	"	"	31.38	0.86	1202	1251	6.19	6.32
314	" $\frac{7}{16}$	"	"	"	33.25	0.81	1238	1310	6.10	6.28
315	" $\frac{1}{2}$	"	"	"	35.13	0.78	1274	1368	6.02	6.24
316	" $\frac{9}{16}$	"	"	"	37.00	0.73	1311	1424	5.95	6.20
317	" $\frac{5}{8}$	"	"	"	38.88	0.69	1347	1479	5.88	6.16
318	" $\frac{11}{16}$	"	"	"	40.75	0.66	1383	1532	5.82	6.13
319	" $\frac{3}{4}$	"	"	"	42.63	0.63	1419	1584	5.76	6.09

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Four Angles and Three Plates.

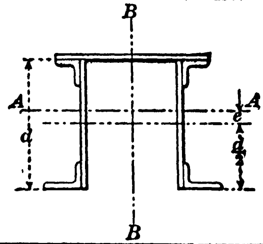
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. c	Moments of Inertia. Axis A–A. I_A	Moments of Inertia. Axis B–B. I_B	Radii of Gyration. Axis A–A. r_A	Radii of Gyration. Axis B–B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
					15″ × 18″ Section. A series.					
*1001	15x3/8	18x7/16	3x3x3/8	4x3x3/8	28.31	1.96	988	1067	5.91	6.14
1002	" 7/16	"	"	"	30.19	1.84	1029	1126	5.84	6.11
1003	" 1/2	"	"	"	32.06	1.73	1070	1184	5.78	6.08
1004	" 9/16	"	"	"	33.94	1.63	1112	1240	5.72	6.05
1005	" 5/8	"	"	"	35.81	1.55	1151	1295	5.67	6.01
1006	" 11/16	"	"	"	37.69	1.47	1191	1348	5.62	5.98
1007	" 3/4	"	"	"	39.56	1.40	1229	1400	5.58	5.95
*1008	15x3/8	18x7/16	3x3x3/8	4x3x7/16	29.09	1.73	1043	1111	5.99	6.18
1009	" 7/16	"	"	"	30.97	1.62	1084	1170	5.92	6.15
1010	" 1/2	"	"	"	32.84	1.53	1123	1228	5.85	6.11
1011	" 9/16	"	"	"	34.72	1.45	1163	1284	5.79	6.08
1012	" 5/8	"	"	"	36.59	1.37	1202	1339	5.73	6.05
1013	" 11/16	"	"	"	38.47	1.30	1241	1392	5.68	6.01
1014	" 3/4	"	"	"	40.34	1.24	1279	1444	5.63	5.98
*1015	15x3/8	18x7/16	3x3x3/8	4x3x1/2	29.85	1.52	1093	1156	6.05	6.22
1016	" 7/16	"	"	"	31.73	1.43	1132	1215	5.97	6.19
1017	" 1/2	"	"	"	33.60	1.35	1171	1273	5.90	6.15
1018	" 9/16	"	"	"	35.48	1.28	1210	1329	5.84	6.12
1019	" 5/8	"	"	"	37.35	1.21	1248	1384	5.78	6.09
1020	" 11/16	"	"	"	39.23	1.15	1286	1437	5.73	6.05
1021	" 3/4	"	"	"	41.10	1.10	1323	1489	5.67	6.02
*1022	15x3/8	18x7/16	3x3x3/8	4x3x9/16	30.59	1.32	1140	1199	6.10	6.26
1023	" 7/16	"	"	"	32.47	1.25	1178	1258	6.02	6.22
1024	" 1/2	"	"	"	34.34	1.18	1216	1316	5.95	6.19
1025	" 9/16	"	"	"	36.22	1.12	1255	1372	5.89	6.16
1026	" 5/8	"	"	"	38.09	1.06	1292	1427	5.83	6.12
1027	" 11/16	"	"	"	39.97	1.01	1329	1480	5.77	6.08
1028	" 3/4	"	"	"	41.84	0.97	1366	1532	5.71	6.05
*1029	15x3/8	18x7/16	3x3x3/8	4x3x5/8	31.31	1.15	1183	1241	6.15	6.30
1030	" 7/16	"	"	"	33.19	1.08	1220	1300	6.06	6.26
1031	" 1/2	"	"	"	35.06	1.02	1257	1358	5.99	6.22
1032	" 9/16	"	"	"	36.94	0.97	1295	1414	5.92	6.19
1033	" 5/8	"	"	"	38.81	0.93	1332	1469	5.86	6.15
1034	" 11/16	"	"	"	40.69	0.88	1368	1522	5.80	6.12
1035	" 3/4	"	"	"	42.56	0.84	1405	1574	5.75	6.08

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

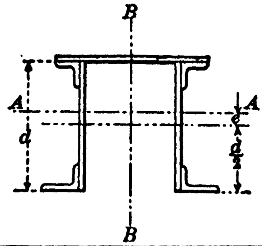
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1036	15x$\frac{3}{8}$	18x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{11}{16}$	32.03	0.98	1223	1284	6.18	6.33
1037	" $\frac{7}{16}$	"	"	"	33.91	0.92	1260	1343	6.10	6.29
1038	" $\frac{1}{2}$	"	"	"	35.78	0.87	1297	1401	6.02	6.25
1039	" $\frac{9}{16}$	"	"	"	37.66	0.83	1334	1457	5.95	6.22
1040	" $\frac{5}{8}$	"	"	"	39.53	0.79	1370	1512	5.89	6.19
1041	" $\frac{11}{16}$	"	"	"	41.41	0.76	1406	1565	5.83	6.15
1042	" $\frac{3}{4}$	"	"	"	43.28	0.72	1442	1617	5.77	6.11
*1043	15x$\frac{3}{8}$	18x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{3}{4}$	32.73	0.82	1259	1327	6.20	6.37
1044	" $\frac{7}{16}$	"	"	"	34.61	0.78	1295	1386	6.12	6.33
1045	" $\frac{1}{2}$	"	"	"	36.48	0.74	1331	1444	6.04	6.29
1046	" $\frac{9}{16}$	"	"	"	38.36	0.70	1368	1500	5.97	6.25
1047	" $\frac{5}{8}$	"	"	"	40.23	0.67	1404	1555	5.90	6.22
1048	" $\frac{11}{16}$	"	"	"	42.11	0.64	1440	1608	5.85	6.18
1049	" $\frac{3}{4}$	"	"	"	43.98	0.61	1475	1660	5.79	6.14
			15" × 18" Section.	B Series.						
1050	15x$\frac{3}{8}$	18x$\frac{3}{8}$	3½x3½x$\frac{3}{8}$	5x3½x$\frac{3}{8}$	29.06	1.50	1035	1042	5.96	5.98
1051	" $\frac{7}{16}$	"	"	"	30.94	1.41	1074	1090	5.89	5.93
1052	" $\frac{1}{2}$	"	"	"	32.81	1.33	1113	1137	5.82	5.88
1053	" $\frac{9}{16}$	"	"	"	34.69	1.26	1151	1183	5.76	5.84
1054	" $\frac{5}{8}$	"	"	"	36.56	1.20	1190	1228	5.70	5.79
1055	" $\frac{11}{16}$	"	"	"	38.44	1.14	1227	1272	5.65	5.75
1056	" $\frac{3}{4}$	"	"	"	40.31	1.08	1265	1315	5.60	5.71
1057	15x$\frac{3}{8}$	18x$\frac{3}{8}$	3½x3½x$\frac{3}{8}$	5x3½x$\frac{7}{16}$	30.02	1.25	1095	1095	6.04	6.04
1058	" $\frac{7}{16}$	"	"	"	31.90	1.18	1133	1143	5.96	5.99
1059	" $\frac{1}{2}$	"	"	"	33.77	1.11	1170	1190	5.89	5.94
1060	" $\frac{9}{16}$	"	"	"	35.65	1.05	1207	1236	5.82	5.89
1061	" $\frac{5}{8}$	"	"	"	37.52	1.00	1245	1281	5.76	5.84
1062	" $\frac{11}{16}$	"	"	"	39.40	0.95	1282	1325	5.70	5.80
1063	" $\frac{3}{4}$	"	"	"	41.27	0.91	1319	1368	5.65	5.75
1064	15x$\frac{3}{8}$	18x$\frac{3}{8}$	3½x3½x$\frac{3}{8}$	5x3½x$\frac{1}{2}$	30.96	1.02	1149	1148	6.09	6.09
1065	" $\frac{7}{16}$	"	"	"	32.84	0.96	1186	1196	6.00	6.03
1066	" $\frac{1}{2}$	"	"	"	34.71	0.91	1222	1243	5.93	5.98
1067	" $\frac{9}{16}$	"	"	"	36.59	0.86	1259	1289	5.86	5.93
1068	" $\frac{5}{8}$	"	"	"	38.46	0.82	1296	1334	5.80	5.88
1069	" $\frac{11}{16}$	"	"	"	40.34	0.78	1332	1378	5.74	5.84
1070	" $\frac{3}{4}$	"	"	"	42.21	0.75	1368	1421	5.69	5.80

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

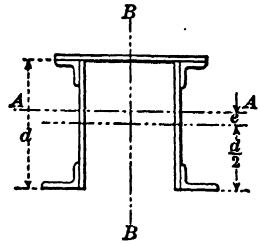
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A–A. I_A	Moments of Inertia. Axis B–B. I_B	Radii of Gyration. Axis A–A. r_A	Radii of Gyration. Axis B–B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
1071	15x3/8	18x3/8	3½x3½x3/8	5x3½x9/16	31.90	0.80	1200	1201	6.13	6.13
1072	" 7/16	"	"	"	33.78	0.75	1236	1249	6.05	6.08
1073	" 1/2	"	"	"	35.65	0.71	1272	1296	5.97	6.03
1074	" 9/16	"	"	"	37.53	0.68	1308	1342	5.90	5.98
1075	" 5/8	"	"	"	39.40	0.65	1344	1387	5.84	5.93
1076	" 11/16	"	"	"	41.28	0.62	1380	1431	5.78	5.89
1077	" 3/4	"	"	"	43.15	0.59	1416	1474	5.72	5.84
1078	15x3/8	18x3/8	3½x3½x3/8	5x3½x5/8	32.80	0.60	1246	1253	6.16	6.18
1079	" 7/16	"	"	"	34.68	0.57	1282	1301	6.08	6.12
1080	" 1/2	"	"	"	36.55	0.54	1317	1348	6.00	6.07
1081	" 9/16	"	"	"	38.43	0.51	1353	1394	5.93	6.02
1082	" 5/8	"	"	"	40.30	0.49	1389	1439	5.87	5.97
1083	" 11/16	"	"	"	42.18	0.47	1425	1483	5.81	5.92
1084	" 3/4	"	"	"	44.05	0.45	1460	1526	5.76	5.88
1085	15x3/8	18x3/8	3½x3½x3/8	5x3½x11/16	33.70	0.41	1289	1305	6.18	6.22
1086	" 7/16	"	"	"	35.58	0.39	1325	1353	6.10	6.16
1087	" 1/2	"	"	"	37.45	0.37	1360	1400	6.02	6.11
1088	" 9/16	"	"	"	39.33	0.35	1395	1446	5.95	6.06
1089	" 5/8	"	"	"	41.20	0.34	1431	1491	5.89	6.01
1090	" 11/16	"	"	"	43.08	0.32	1467	1535	5.83	5.96
1091	" 3/4	"	"	"	44.95	0.31	1502	1578	5.78	5.92
1092	15x3/8	18x3/8	3½x3½x3/8	5x3½x3/4	34.58	0.25	1326	1358	6.19	6.26
1093	" 7/16	"	"	"	36.46	0.23	1361	1406	6.11	6.20
1094	" 1/2	"	"	"	38.33	0.22	1396	1453	6.03	6.15
1095	" 9/16	"	"	"	40.21	0.21	1431	1499	5.96	6.10
1096	" 5/8	"	"	"	42.08	0.20	1467	1544	5.90	6.05
1097	" 11/16	"	"	"	43.96	0.19	1502	1588	5.84	6.00
1098	" 3/4	"	"	"	45.83	0.18	1537	1631	5.79	5.96
				15″ × 19″ Section. A Series.						
*1099	15x3/8	19x7/16	3x3x3/8	4x3x3/8	28.75	2.04	1002	1240	5.91	6.57
1100	" 7/16	"	"	"	30.63	1.92	1044	1310	5.84	6.54
1101	" 1/2	"	"	"	32.50	1.81	1086	1378	5.78	6.51
1102	" 9/16	"	"	"	34.38	1.71	1128	1445	5.73	6.48
1103	" 5/8	"	"	"	36.25	1.62	1168	1510	5.68	6.45
1104	" 11/16	"	"	"	38.13	1.54	1207	1574	5.63	6.43
1105	" 3/4	"	"	"	40.00	1.47	1247	1637	5.59	6.40

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

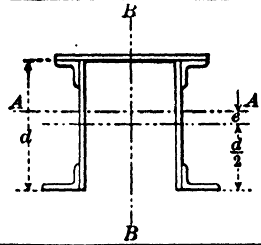
Section Number.	Plates.		Angles.		Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.			Axis A–A.	Axis B–B.	Axis A–A.	Axis B–B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1106	15x$\frac{3}{8}$	19x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{7}{16}$	29.53	1.81	1059	1291	5.99	6.61
1107	" $\frac{7}{16}$	"	"	"	31.41	1.71	1100	1361	5.92	6.58
1108	" $\frac{1}{2}$	"	"	"	33.28	1.61	1140	1429	5.85	6.55
1109	" $\frac{9}{16}$	"	"	"	35.16	1.52	1180	1496	5.79	6.52
1110	" $\frac{5}{8}$	"	"	"	37.03	1.45	1219	1561	5.74	6.49
1111	" $\frac{11}{16}$	"	"	"	38.91	1.38	1258	1625	5.69	6.46
1112	" $\frac{3}{4}$	"	"	"	40.78	1.31	1297	1688	5.64	6.43
*1113	15x$\frac{3}{8}$	19x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{1}{2}$	30.29	1.61	1110	1341	6.05	6.65
1114	" $\frac{7}{16}$	"	"	"	32.17	1.51	1149	1411	5.98	6.62
1115	" $\frac{1}{2}$	"	"	"	34.04	1.43	1188	1479	5.91	6.59
1116	" $\frac{9}{16}$	"	"	"	35.92	1.36	1228	1546	5.85	6.56
1117	" $\frac{5}{8}$	"	"	"	37.79	1.29	1266	1611	5.79	6.53
1118	" $\frac{11}{16}$	"	"	"	39.67	1.23	1304	1675	5.73	6.50
1119	" $\frac{3}{4}$	"	"	"	41.54	1.17	1342	1738	5.68	6.47
*1120	15x$\frac{3}{8}$	19x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{9}{16}$	31.03	1.41	1158	1390	6.11	6.69
1121	" $\frac{7}{16}$	"	"	"	32.91	1.33	1196	1460	6.03	6.66
1122	" $\frac{1}{2}$	"	"	"	34.78	1.26	1235	1528	5.96	6.63
1123	" $\frac{9}{16}$	"	"	"	36.66	1.20	1273	1595	5.89	6.60
1124	" $\frac{5}{8}$	"	"	"	38.53	1.14	1311	1660	5.83	6.57
1125	" $\frac{11}{16}$	"	"	"	40.41	1.09	1348	1724	5.77	6.53
1126	" $\frac{3}{4}$	"	"	"	42.28	1.04	1385	1787	5.72	6.50
*1127	15x$\frac{3}{8}$	19x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{5}{8}$	31.75	1.24	1201	1437	6.15	6.73
1128	" $\frac{7}{16}$	"	"	"	33.63	1.17	1239	1507	6.07	6.70
1129	" $\frac{1}{2}$	"	"	"	35.50	1.11	1277	1575	6.00	6.66
1130	" $\frac{9}{16}$	"	"	"	37.38	1.05	1315	1642	5.93	6.63
1131	" $\frac{5}{8}$	"	"	"	39.25	1.00	1352	1707	5.87	6.60
1132	" $\frac{11}{16}$	"	"	"	41.13	0.96	1388	1771	5.81	6.56
1133	" $\frac{3}{4}$	"	"	"	43.00	0.91	1425	1834	5.76	6.53
*1134	15x$\frac{3}{8}$	19x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{11}{16}$	32.47	1.07	1243	1486	6.19	6.76
1135	" $\frac{7}{16}$	"	"	"	34.35	1.01	1280	1556	6.10	6.73
1136	" $\frac{1}{2}$	"	"	"	36.22	0.96	1317	1624	6.03	6.70
1137	" $\frac{9}{16}$	"	"	"	38.10	0.91	1354	1691	5.96	6.66
1138	" $\frac{5}{8}$	"	"	"	39.97	0.87	1391	1756	5.90	6.63
1139	" $\frac{11}{16}$	"	"	"	41.85	0.83	1427	1820	5.84	6.60
1140	" $\frac{3}{4}$	"	"	"	43.72	0.79	1463	1883	5.79	6.56

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Four Angles and Three Plates.

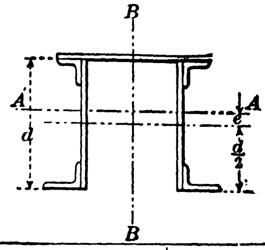
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1141	15x$\frac{3}{8}$	19x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{3}{4}$	33.17	0.92	1279	1535	6.21	6.80
1142	" $\frac{7}{16}$	"	"	"	35.05	0.87	1316	1605	6.13	6.77
1143	" $\frac{1}{2}$	"	"	"	36.92	0.82	1352	1673	6.05	6.73
1144	" $\frac{9}{16}$	"	"	"	38.80	0.78	1388	1740	5.98	6.70
1145	" $\frac{5}{8}$	"	"	"	40.67	0.75	1425	1805	5.92	6.66
1146	" $\frac{11}{16}$	"	"	"	42.55	0.71	1461	1869	5.86	6.63
1147	" $\frac{3}{4}$	"	"	"	44.42	0.68	1497	1932	5.81	6.59
				15″ × 19″ Section.	B Series.					
1148	15x$\frac{3}{8}$	19x$\frac{7}{16}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{3}{8}$	30.62	1.83	1094	1250	5.98	6.39
1149	" $\frac{7}{16}$	"	"	"	32.50	1.72	1136	1308	5.91	6.34
1150	" $\frac{1}{2}$	"	"	"	34.37	1.63	1176	1365	5.85	6.30
1151	" $\frac{9}{16}$	"	"	"	36.25	1.55	1215	1421	5.79	6.26
1152	" $\frac{5}{8}$	"	"	"	38.12	1.47	1255	1476	5.73	6.22
1153	" $\frac{11}{16}$	"	"	"	40.00	1.40	1294	1530	5.68	6.18
1154	" $\frac{3}{4}$	"	"	"	41.87	1.34	1333	1583	5.64	6.14
1155	15x$\frac{3}{8}$	19x$\frac{7}{16}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{7}{16}$	31.58	1.58	1160	1310	6.06	6.44
1156	" $\frac{7}{16}$	"	"	"	33.46	1.49	1200	1368	5 98	6.39
1157	" $\frac{1}{2}$	"	"	"	35.33	1.41	1239	1425	5.92	6.35
1158	" $\frac{9}{16}$	"	"	"	37.21	1.34	1277	1481	5.86	6.31
1159	" $\frac{5}{8}$	"	"	"	39.08	1.27	1317	1536	5.80	6.27
1160	" $\frac{11}{16}$	"	"	"	40.96	1.21	1355	1590	5.75	6.23
1161	" $\frac{3}{4}$	"	"	"	42.83	1.16	1392	1643	5.70	6.19
1162	15x$\frac{3}{8}$	19x$\frac{7}{16}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{1}{2}$	32.52	1.35	1218	1371	6.12	6.49
1163	" $\frac{7}{16}$	"	"	"	34.40	1.27	1256	1429	6.04	6.44
1164	" $\frac{1}{2}$	"	"	"	36.27	1.21	1294	1486	5.97	6.40
1165	" $\frac{9}{16}$	"	"	"	38.15	1.15	1332	1542	5.91	6.36
1166	" $\frac{5}{8}$	"	"	"	40.02	1.09	1370	1597	5.85	6.32
1167	" $\frac{11}{16}$	"	"	"	41.90	1.04	1407	1651	5.79	6.28
1168	" $\frac{3}{4}$	"	"	"	43.77	1.00	1414	1704	5.74	6.24
1169	15x$\frac{3}{8}$	19x$\frac{7}{16}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{9}{16}$	33.46	1.13	1274	1431	6.17	6.54
1170	" $\frac{7}{16}$	"	"	"	35.34	1.07	1311	1489	6.09	6.49
1171	" $\frac{1}{2}$	"	"	"	37.11	1.02	1348	1546	6.02	6.45
1172	" $\frac{9}{16}$	"	"	"	38.99	0.97	1385	1602	5.96	6.41
1173	" $\frac{5}{8}$	"	"	"	40.86	0.92	1423	1657	5.90	6.37
1174	" $\frac{11}{16}$	"	"	"	42.74	0.88	1460	1711	5.84	6.33
1175	" $\frac{3}{4}$	"	"	"	44.61	0.85	1496	1764	5.79	6.29

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Four Angles and Three Plates.

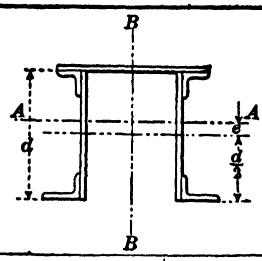
Section Number	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
1176	15x3/8	19x7/16	3½x3½x3/8	5x3½x5/8	34.36	0.93	1325	1490	6.21	6.59
1177	" 7/16	"	"	"	36.24	0.88	1362	1548	6.13	6.53
1178	" 1/2	"	"	"	38.11	0.84	1398	1605	6.06	6.48
1179	" 9/16	"	"	"	39.99	0.80	1434	1661	5.99	6.44
1180	" 5/8	"	"	"	41.86	0.76	1472	1716	5.93	6.40
1181	" 11/16	"	"	"	43.74	0.73	1508	1770	5.87	6.36
1182	" 3/4	"	"	"	45.61	0.70	1544	1823	5.82	6.32
1183	15x3/8	19x7/16	3½x3½x3/8	5x3½x11/16	35.26	0.74	1372	1549	6.24	6.63
1184	" 7/16	"	"	"	37.14	0.70	1408	1607	6.16	6.58
1185	" 1/2	"	"	"	39.01	0.67	1444	1664	6.08	6.53
1186	" 9/16	"	"	"	40.89	0.64	1479	1720	6.01	6.48
1187	" 5/8	"	"	"	42.76	0.61	1516	1775	5.95	6.44
1188	" 11/16	"	"	"	44.64	0.59	1552	1829	5.89	6.40
1189	" 3/4	"	"	"	46.51	0.56	1587	1882	5.84	6.36
1190	15x3/8	19x7/16	3½x3½x3/8	5x3½x3/4	36.14	0.58	1413	1609	6.25	6.67
1191	" 7/16	"	"	"	38.02	0.55	1448	1667	6.16	6.62
1192	" 1/2	"	"	"	39.89	0.52	1484	1724	6.09	6.57
1193	" 9/16	"	"	"	41.77	0.50	1520	1780	6.03	6.52
1194	" 5/8	"	"	"	43.64	0.48	1556	1835	5.97	6.48
1195	" 11/16	"	"	"	45.52	0.46	1591	1889	5.91	6.44
1196	" 3/4	"	"	"	47.39	0.44	1627	1942	5.86	6.40
			16" × 19" Section.	A Series.						
*1197	16x3/8	19x7/16	3x3x3/8	4x3x3/8	29.49	2.12	1165	1270	6.28	6.56
1198	" 7/16	"	"	"	31.49	1.99	1216	1344	6.21	6.53
1199	" 1/2	"	"	"	33.49	1.87	1265	1417	6.15	6.51
1200	" 9/16	"	"	"	35.49	1.76	1315	1488	6.09	6.48
1201	" 5/8	"	"	"	37.49	1.67	1364	1558	6.04	6.45
1202	" 11/16	"	"	"	39.49	1.58	1412	1626	5.98	6.42
1203	" 3/4	"	"	"	41.49	1.51	1459	1693	5.93	6.39
*1204	16x3/8	19x7/16	3x3x3/8	4x3x7/16	30.27	1.88	1229	1321	6.37	6.60
1205	" 7/16	"	"	"	32.27	1.77	1278	1395	6.29	6.57
1206	" 1/2	"	"	"	34.27	1.66	1326	1468	6.22	6.54
1207	" 9/16	"	"	"	36.27	1.57	1374	1539	6.15	6.51
1208	" 5/8	"	"	"	38.27	1.49	1422	1609	6.09	6.48
1209	" 11/16	"	"	"	40.27	1.42	1469	1677	6.04	6.45
1210	" 3/4	"	"	"	42.27	1.35	1515	1744	5.99	6.42

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

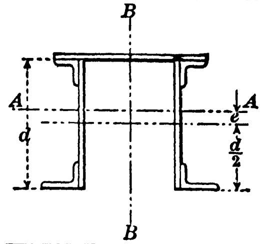
Section Number	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
					16″ × 19″ Section. A Series.					
*1211	16x3/8	19x7/16	3x3x3/8	4x3x1/2	31.03	1.67	1287	1371	6.45	6.65
1212	" 7/16	"	"	"	33.03	1.57	1335	1445	6.36	6.62
1213	" 1/2	"	"	"	35.03	1.48	1382	1518	6.28	6.58
1214	" 9/16	"	"	"	37.03	1.40	1429	1589	6.21	6.55
1215	" 5/8	"	"	"	39.03	1.32	1476	1659	6.15	6.52
1216	" 11/16	"	"	"	41.03	1.26	1522	1727	6.09	6.49
1217	" 3/4	"	"	"	43.03	1.20	1567	1794	6.04	6.46
*1218	16x3/8	19x7/16	3x3x3/8	4x3x9/16	31.77	1.46	1342	1420	6.50	6.69
1219	" 7/16	"	"	"	33.77	1.38	1389	1494	6.41	6.65
1220	" 1/2	"	"	"	35.77	1.30	1435	1567	6.33	6.62
1221	" 9/16	"	"	"	37.77	1.23	1481	1638	6.26	6.58
1222	" 5/8	"	"	"	39.77	1.17	1527	1708	6.19	6.55
1223	" 11/16	"	"	"	41.77	1.11	1572	1776	6.13	6.52
1224	" 3/4	"	"	"	43.77	1.06	1617	1843	6.08	6.49
*1225	16x3/8	19x7/16	3x3x3/8	4x3x5/8	32.49	1.28	1392	1467	6.55	6.72
1226	" 7/16	"	"	"	34.49	1.20	1438	1541	6.46	6.68
1227	" 1/2	"	"	"	36.49	1.14	1483	1614	6.37	6.65
1228	" 9/16	"	"	"	38.49	1.08	1528	1685	6.30	6.62
1229	" 5/8	"	"	"	40.49	1.03	1573	1755	6.23	6.58
1230	" 11/16	"	"	"	42.49	0.98	1618	1823	6.17	6.55
1231	" 3/4	"	"	"	44.49	0.93	1662	1890	6 11	6.52
*1232	16x3/8	19x7/16	3x3x3/8	4x3x11/16	33.21	1.10	1439	1516	6.58	6.76
1233	" 7/16	"	"	"	35.21	1.04	1484	1590	6.49	6.72
1234	" 1/2	"	"	"	37.21	0 98	1528	1663	6.41	6.68
1235	" 9/16	"	"	"	39.21	0.93	1573	1734	6.33	6.65
1236	" 5/8	"	"	"	41.21	0.89	1617	1804	6.26	6.62
1237	" 11/16	"	"	"	43.21	0.85	1662	1872	6.20	6.58
1238	" 3/4	"	"	"	45.21	0.81	1705	1939	6.14	6.55
*1239	16x3/8	19x7/16	3x3x3/8	4x3x3/4	33.91	0.94	1481	1565	6.61	6.79
1240	" 7/16	"	"	"	35.91	0.89	1526	1639	6.52	6.76
1241	" 1/2	"	"	"	37.91	0.84	1569	1712	6.43	6.72
1242	" 9/16	"	"	"	39.91	0.80	1614	1783	6.36	6.68
1243	" 5/8	"	"	"	41.91	0.76	1658	1853	6.29	6.65
1244	" 11/16	"	"	"	43.91	0.73	1702	1921	6.23	6.61
1245	" 3/4	"	"	"	45.91	0.70	1745	1988	6.17	6.58

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

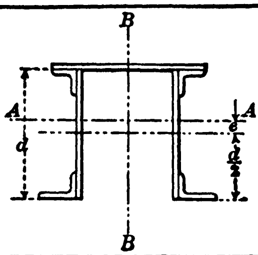
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches^2.	Inches.	Inches^4.	Inches^4.	Inches.	Inches.
				16″ × 19″ Section. B Series.						
*1246	16x3/8	19x7/16	3½x3½x3/8	5x3½x3/8	31.37	1.90	1271	1275	6.36	6.37
1247	" 7/16	"	"	"	33.37	1.79	1320	1337	6.29	6.33
1248	" 1/2	"	"	"	35.37	1.69	1368	1398	6.22	6.28
1249	" 9/16	"	"	"	37.37	1.60	1417	1458	6.15	6.24
1250	" 5/8	"	"	"	39.37	1.52	1464	1516	6.10	6.20
1251	" 11/16	"	"	"	41.37	1.44	1511	1573	6.05	6.16
1252	" 3/4	"	"	"	43.37	1.37	1558	1629	6.00	6.13
*1253	16x3/8	19x7/16	3½x3½x3/8	5x3½x7/16	32.33	1.64	1345	1335	6.45	6.42
1254	" 7/16	"	"	"	34.33	1.54	1393	1397	6.37	6.38
1255	" 1/2	"	"	"	36.33	1.46	1440	1458	6.30	6.33
1256	" 9/16	"	"	"	38.33	1.38	1487	1518	6.23	6.29
1257	" 5/8	"	"	"	40.33	1.31	1534	1576	6.17	6.25
1258	" 11/16	"	"	"	42.33	1.25	1579	1633	6.11	6.21
1259	" 3/4	"	"	"	44.33	1.19	1625	1689	6.05	6.17
*1260	16x3/8	19x7/16	3½x3½x3/8	5x3½x1/2	33.27	1.40	1412	1396	6.51	6.48
1261	" 7/16	"	"	"	35.27	1.32	1459	1458	6.42	6.42
1262	" 1/2	"	"	"	37.27	1.25	1504	1519	6.35	6.38
1263	" 9/16	"	"	"	39.27	1.18	1550	1579	6.28	6.34
1264	" 5/8	"	"	"	41.27	1.13	1595	1637	6.21	6.30
1265	" 11/16	"	"	"	43.27	1.08	1640	1694	6.15	6.26
1266	" 3/4	"	"	"	45.27	1.03	1685	1750	6.10	6.22
*1267	16x3/8	19x7/16	3½x3½x3/8	5x3½x9/16	34.21	1.17	1475	1456	6.57	6.52
1268	" 7/16	"	"	"	36.21	1.10	1521	1518	6.48	6.47
1269	" 1/2	"	"	"	38.21	1.05	1565	1579	6.39	6.42
1270	" 9/16	"	"	"	40.21	1.00	1610	1639	6.32	6.38
1271	" 5/8	"	"	"	42.21	0.95	1655	1697	6.26	6.34
1272	" 11/16	"	"	"	44.21	0.91	1699	1754	6.20	6.30
1273	" 3/4	"	"	"	46.21	0.87	1743	1810	6.14	6.26
*1274	16x3/8	19x7/16	3½x3½x3/8	5x3½x5/8	35.11	0.96	1534	1514	6.61	6.57
1275	" 7/16	"	"	"	37.11	0.91	1578	1576	6.52	6.51
1276	" 1/2	"	"	"	39.11	0.85	1622	1637	6.44	6.46
1277	" 9/16	"	"	"	41.11	0.82	1666	1697	6.36	6.42
1278	" 5/8	"	"	"	43.11	0.78	1711	1755	6.29	6.38
1279	" 11/16	"	"	"	45.11	0.75	1754	1812	6.23	6.34
1280	" 3/4	"	"	"	47.11	0.72	1798	1868	6.17	6.30

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

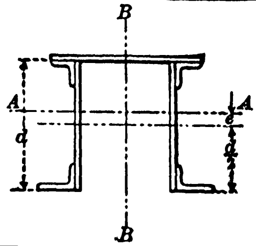
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1281	16x3/8	19x7/16	3½x3½x3/8	5x3½x11/16	36.01	0.77	1586	1573	6.64	6.60
1282	“ 7/16	“	“	“	38.01	0.73	1630	1635	6.55	6.56
1283	“ 1/2	“	“	“	40.01	0.69	1673	1696	6.47	6.51
1284	“ 9/16	“	“	“	42.01	0.66	1717	1756	6.39	6.46
1285	“ 5/8	“	“	“	44.01	0.63	1761	1814	6.32	6.42
1286	“ 11/16	“	“	“	46.01	0.60	1803	1871	6.26	6.37
1287	“ 3/4	“	“	“	48.01	0.57	1847	1927	6.20	6.33
*1288	16x3/8	19x7/16	3½x3½x3/8	5x3½x3/4	36.89	0.59	1632	1634	6.65	6.65
1289	“ 7/16	“	“	“	38.89	0.56	1678	1694	6.56	6.59
1290	“ 1/2	“	“	“	40.89	0.53	1720	1755	6.48	6.55
1291	“ 9/16	“	“	“	42.89	0.51	1764	1815	6.41	6.50
1292	“ 5/8	“	“	“	44.89	0.48	1807	1873	6.34	6.46
1293	“ 11/16	“	“	“	46.89	0.46	1850	1930	6.28	6.42
1294	“ 3/4	“	“	“	48.89	0.44	1893	1986	6.22	6.37
			16″ × 20″ Section.	A Series.						
*1295	16x3/8	20x7/16	3x3x3/8	4x3x3/8	29.93	2.21	1180	1463	6.28	6.99
1296	“ 7/16	“	“	“	31.93	2.07	1232	1550	6.21	6.97
1297	“ 1/2	“	“	“	33.93	1.95	1282	1635	6.15	6.94
1298	“ 9/16	“	“	“	35.93	1.84	1332	1719	6.09	6.92
1299	“ 5/8	“	“	“	37.93	1.74	1382	1801	6.04	6 89
1300	“ 11/16	“	“	“	39.93	1.65	1431	1881	5.99	6.86
1301	“ 3/4	“	“	“	41.93	1.58	1478	1959	5.94	6.84
*1302	16x3/8	20x7/16	3x3x3/8	4x3x7/16	30.71	1.97	1246	1519	6.37	7.04
1303	“ 7/16	“	“	“	32.71	1.85	1297	1606	6.30	7.01
1304	“ 1/2	“	“	“	34.71	1.75	1346	1691	6.23	6.98
1305	“ 9/16	“	“	“	36.71	1.65	1394	1775	6.16	6.95
1306	“ 5/8	“	“	“	38.71	1.57	1442	1857	6.10	6.93
1307	“ 11/16	“	“	“	40.71	1.49	1490	1937	6.05	6.90
1308	“ 3/4	“	“	“	42.71	1.42	1536	2015	6.00	6.87
*1309	16x3/8	20x7/16	3x3x3/8	4x3x1/2	31.47	1.76	1306	1576	6.44	7.08
1310	“ 7/16	“	“	“	33.47	1.65	1355	1663	6.36	7.05
1311	“ 1/2	“	“	“	35.47	1.56	1402	1748	6.29	7.02
1312	“ 9/16	“	“	“	37.47	1.48	1449	1832	6.22	6.99
1313	“ 5/8	“	“	“	39.47	1.40	1496	1914	6.16	6.96
1314	“ 11/16	“	“	“	41.47	1.33	1543	1994	6.10	6.93
1315	“ 3/4	“	“	“	43.47	1.27	1589	2072	6.05	6.90

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

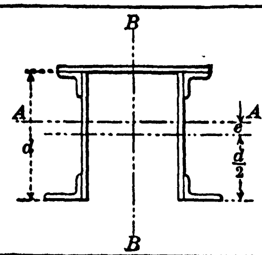
Section Number.	Plates.		Angles.		Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.			Axis A-A.	Axis B-B.	Axis A-A.	Axis B-B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1316	16x$\frac{3}{8}$	20x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{9}{16}$	32.21	1.55	1361	1631	6.50	7.12
1317	" $\frac{7}{16}$	"	"	"	34.21	1.46	1409	1718	6.42	7.09
1318	" $\frac{1}{2}$	"	"	"	36.21	1.38	1455	1803	6.34	7.06
1319	" $\frac{9}{16}$	"	"	"	38.21	1.31	1501	1887	6.27	7.03
1320	" $\frac{5}{8}$	"	"	"	40.21	1.25	1548	1969	6.20	7.00
1321	" $\frac{11}{16}$	"	"	"	42.21	1.19	1504	2049	6.15	6.97
1322	" $\frac{3}{4}$	"	"	"	44.21	1.13	1638	2127	6.09	6.94
*1323	16x$\frac{3}{8}$	20x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{5}{8}$	32.93	1.37	1412	1685	6.55	7.16
1324	" $\frac{7}{16}$	"	"	"	34.93	1.29	1459	1772	6.46	7.12
1325	" $\frac{1}{2}$	"	"	"	36.93	1.22	1504	1857	6.38	7.09
1326	" $\frac{9}{16}$	"	"	"	38.93	1.16	1550	1941	6.31	7.06
1327	" $\frac{5}{8}$	"	"	"	40.93	1.10	1595	2023	6.24	7.03
1328	" $\frac{11}{16}$	"	"	"	42.93	1.05	1641	2103	6.18	7.00
1329	" $\frac{3}{4}$	"	"	"	44.93	1.00	1685	2181	6.13	6.97
*1330	16x$\frac{3}{8}$	20x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{11}{16}$	33.65	1.19	1461	1739	6.59	7.19
1331	" $\frac{7}{16}$	"	"	"	35.65	1.12	1507	1826	6.50	7.16
1332	" $\frac{1}{2}$	"	"	"	37.65	1.06	1551	1911	6.42	7.12
1333	" $\frac{9}{16}$	"	"	"	39.65	1.01	1596	1995	6.35	7.09
1334	" $\frac{5}{8}$	"	"	"	41.65	0.96	1641	2077	6.28	7.06
1335	" $\frac{11}{16}$	"	"	"	43.65	0.92	1686	2157	6.22	7.03
1336	" $\frac{3}{4}$	"	"	"	45.65	0.88	1730	2235	6.16	7.00
*1337	16x$\frac{3}{8}$	20x$\frac{7}{16}$	3x3x$\frac{3}{8}$	4x3x$\frac{3}{4}$	34.35	1.03	1504	1794	6.62	7.23
1338	" $\frac{7}{16}$	"	"	"	36.35	0.98	1549	1881	6.53	7.19
1339	" $\frac{1}{2}$	"	"	"	38.35	0.93	1593	1966	6.45	7.16
1340	" $\frac{9}{16}$	"	"	"	40.35	0.88	1638	2050	6.37	7.13
1341	" $\frac{5}{8}$	"	"	"	42.35	0.84	1682	2132	6.30	7.10
1342	" $\frac{11}{16}$	"	"	"	44.35	0.80	1727	2212	6.24	7.06
1343	" $\frac{3}{4}$	"	"	"	46.35	0.77	1770	2290	6.18	7.03
16″ X 20″ Section. B Series.										
*1344	16x$\frac{3}{8}$	20x$\frac{7}{16}$	3½x3½x$\frac{3}{8}$	5x3½x$\frac{3}{8}$	31.81	1.99	1288	1473	6.36	6.80
1345	" $\frac{7}{16}$	"	"	"	33.81	1.87	1339	1547	6.28	6.76
1346	" $\frac{1}{2}$	"	"	"	35.81	1.76	1388	1620	6.22	6.72
1347	" $\frac{9}{16}$	"	"	"	37.81	1.67	1437	1691	6.16	6.68
1348	" $\frac{5}{8}$	"	"	"	39.81	1.59	1485	1761	6.10	6.64
1349	" $\frac{11}{16}$	"	"	"	41.81	1.51	1532	1829	6.05	6.61
1350	" $\frac{3}{4}$	"	"	"	43.81	1.44	1579	1896	6.00	6.58

* Spacing of rivet lines of web greater than 30 X thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

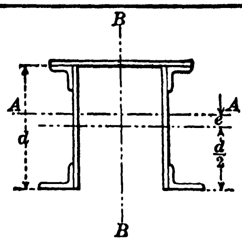
Section Number.	Plates.		Angles.		Gross Area.	Eccentricity	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.			Axis A-A.	Axis B-B.	Axis A-A.	Axis B-B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1351	16x$\frac{3}{8}$	20x$\frac{7}{16}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{7}{16}$	32.77	1.72	1364	1541	6.45	6.85
1352	" $\frac{7}{16}$	"	"	"	34.77	1.62	1412	1615	6.37	6.81
1353	" $\frac{1}{2}$	"	"	"	36.77	1.54	1459	1688	6.30	6.77
1354	" $\frac{9}{16}$	"	"	"	38.77	1.46	1506	1759	6.23	6.74
1355	" $\frac{5}{8}$	"	"	"	40.77	1.39	1553	1829	6.17	6.70
1356	" $\frac{11}{16}$	"	"	"	42.77	1.32	1599	1897	6.11	6.66
1357	" $\frac{3}{4}$	"	"	"	44.77	1.26	1646	1964	6.06	6.62
*1358	16x$\frac{3}{8}$	20x$\frac{7}{16}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{1}{2}$	33.71	1.49	1431	1609	6.51	6.91
1359	" $\frac{7}{16}$	"	"	"	35.71	1.40	1479	1683	6.43	6.86
1360	" $\frac{1}{2}$	"	"	"	37.71	1.33	1525	1756	6.35	6.82
1361	" $\frac{9}{16}$	"	"	"	39.71	1.26	1571	1827	6.29	6.78
1362	" $\frac{5}{8}$	"	"	"	41.71	1.20	1617	1897	6.22	6.74
1363	" $\frac{11}{16}$	"	"	"	43.71	1.15	1661	1965	6.16	6.70
1364	" $\frac{3}{4}$	"	"	"	45.71	1.10	1707	2032	6.11	6.66
*1365	16x$\frac{3}{8}$	20x$\frac{7}{16}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{9}{16}$	34.65	1.26	1497	1677	6.57	6.96
1366	" $\frac{7}{16}$	"	"	"	36.65	1.19	1543	1751	6.48	6.91
1367	" $\frac{1}{2}$	"	"	"	38.65	1.13	1588	1824	6.41	6.87
1368	" $\frac{9}{16}$	"	"	"	40.65	1.07	1633	1895	6.34	6.83
1369	" $\frac{5}{8}$	"	"	"	42.65	1.02	1678	1965	6.27	6.79
1370	" $\frac{11}{16}$	"	"	"	44.65	0.98	1722	2033	6.21	6.75
1371	" $\frac{3}{4}$	"	"	"	46.65	0.94	1767	2100	6.15	6.71
*1372	16x$\frac{3}{8}$	20x$\frac{7}{16}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{5}{8}$	35.55	1.05	1556	1742	6.61	7.00
1373	" $\frac{7}{16}$	"	"	"	37.55	0.99	1600	1816	6.53	6.95
1374	" $\frac{1}{2}$	"	"	"	39.55	0.94	1644	1889	6.45	6.91
1375	" $\frac{9}{16}$	"	"	"	41.55	0.90	1698	1960	6.37	6.87
1376	" $\frac{5}{8}$	"	"	"	43.55	0.86	1733	2030	6.31	6.83
1377	" $\frac{11}{16}$	"	"	"	45.55	0.82	1777	2098	6.24	6.78
1378	" $\frac{3}{4}$	"	"	"	47.55	0.78	1822	2165	6.19	6.74
*1379	16x$\frac{3}{8}$	20x$\frac{7}{16}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{11}{16}$	36.45	0.86	1610	1808	6.64	7.04
1380	" $\frac{7}{16}$	"	"	"	38.45	0.81	1655	1882	6.56	6.99
1381	" $\frac{1}{2}$	"	"	"	40.45	0.77	1698	1955	6.48	6.95
1382	" $\frac{9}{16}$	"	"	"	42.45	0.73	1742	2026	6.41	6.91
1383	" $\frac{5}{8}$	"	"	"	44.45	0.70	1786	2096	6.34	6.87
1384	" $\frac{11}{16}$	"	"	"	46.45	0.67	1829	2164	6.28	6.83
1385	" $\frac{3}{4}$	"	"	"	48.45	0.64	1873	2232	6.22	6.79

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

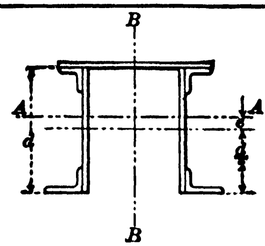
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches	Inches⁴.	Inches⁴.	Inches.	Inches.
*1386	16x3/8	20x7/16	3½x3½x3/8	5x3½x3/4	37.33	0.68	1660	1875	6.67	7.09
1387	“ 7/16	“	“	“	39.33	0.64	1704	1949	6.58	7.03
1388	“ 1/2	“	“	“	41.33	0.61	1747	2022	6.50	6.99
1389	“ 9/16	“	“	“	43.33	0.58	1790	2093	6.42	6.94
1390	“ 5/8	“	“	“	45.33	0.56	1834	2163	6.36	6.90
1391	“ 11/16	“	“	“	47.33	0.53	1876	2231	6.30	6.86
1392	“ 3/4	“	“	“	49.33	0.51	1920	2298	6.24	6.83
			18″ × 21″ Section.	A Series.						
*1393	18x7/16	21x1/2	3x3x3/8	4x3x3/8	35.43	2.56	1712	1912	6.95	7.35
1394	“ 1/2	“	“	“	37.68	2.40	1787	2023	6 89	7.33
1395	“ 9/16	“	“	“	39.93	2.27	1860	2132	6.82	7.31
1396	“ 5/8	“	“	“	42.18	2.15	1931	2239	6.77	7.29
1397	“ 11/16	“	“	“	44.43	2.04	2002	2345	6.72	7.27
1398	“ 3/4	“	“	“	46.68	1.94	2072	2449	6.66	7.24
*1399	18x7/16	21x1/2	3x3x3/8	4x3x7/16	36.21	2.33	1799	1975	7.05	7.39
1400	“ 1/2	“	“	“	38.46	2.19	1871	2086	6.97	7.37
1401	“ 9/16	“	“	“	40.71	2.07	1942	2195	6.91	7.35
1402	“ 5/8	“	“	“	42.96	1.96	2012	2302	6.85	7.32
1403	“ 11/16	“	“	“	45.21	1.86	2081	2408	6.79	7.30
1404	“ 3/4	“	“	“	47.46	1.78	2149	2512	6.73	7.28
*1405	18x7/16	21x1/2	3x3x3/8	4x3x1/2	36.97	2.12	1878	2039	7.13	7.43
1406	“ 1/2	“	“	“	39.22	2.00	1948	2150	7.05	7.41
1407	“ 9/16	“	“	“	41.47	1.89	2018	2259	6.98	7.38
1408	“ 5/8	“	“	“	43.72	1.79	2086	2366	6.91	7.36
1409	“ 11/16	“	“	“	45.97	1.70	2154	2472	6.85	7.33
1410	“ 3/4	“	“	“	48.22	1.62	2221	2576	6.79	7.31
*1411	18x7/16	21x1/2	3x3x3/8	4x3x9/16	37.71	1.92	1952	2100	7.20	7.46
1412	“ 1/2	“	“	“	39.96	1.81	2021	2211	7.11	7.44
1413	“ 9/16	“	“	“	42 21	1 72	2089	2320	7.03	7.42
1414	“ 5/8	“	“	“	44 46	1.63	2155	2427	6.96	7.39
1415	“ 11/16	“	“	“	46.71	1.55	2222	2533	6.90	7.36
1416	“ 3/4	“	“	“	48.96	1.48	2288	2637	6.84	7.34

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

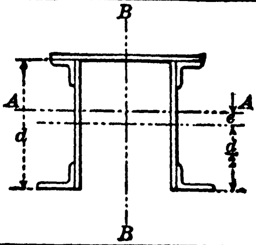
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1417	18x$\frac{7}{16}$	21x$\frac{1}{2}$	3x3x$\frac{3}{8}$	4x3x$\frac{5}{8}$	38.43	1.74	2021	2160	7.25	7.50
1418	" $\frac{1}{2}$	"	"	"	40.68	1.64	2088	2271	7.17	7.47
1419	" $\frac{9}{16}$	"	"	"	42.93	1.55	2154	2380	7.09	7.45
1420	" $\frac{5}{8}$	"	"	"	45.18	1.48	2220	2487	7.01	7.42
1421	" $\frac{11}{16}$	"	"	"	47.43	1.41	2286	2593	6.94	7.40
1422	" $\frac{3}{4}$	"	"	"	49.68	1.34	2351	2697	6.88	7.37
*1423	18x$\frac{7}{16}$	21x$\frac{1}{2}$	3x3x$\frac{3}{8}$	4x3x$\frac{11}{16}$	39.15	1.56	2087	2221	7.30	7.53
1424	" $\frac{1}{2}$	"	"	"	41.40	1.47	2153	2332	7.21	7.51
1425	" $\frac{9}{16}$	"	"	"	43.65	1.40	2219	2441	7.13	7.48
1426	" $\frac{5}{8}$	"	"	"	45.90	1.33	2283	2548	7.05	7.45
1427	" $\frac{11}{16}$	"	"	"	48.15	1.27	2348	2654	6.98	7.43
1428	" $\frac{3}{4}$	"	"	"	50.40	1.21	2412	2758	6.92	7.40
*1429	18x$\frac{7}{16}$	21x$\frac{1}{2}$	3x3x$\frac{3}{8}$	4x3x$\frac{3}{4}$	39.85	1.40	2146	2282	7.34	7.57
1430	" $\frac{1}{2}$	"	"	"	42.10	1.32	2212	2393	7.25	7.54
1431	" $\frac{9}{16}$	"	"	"	44.35	1.25	2276	2502	7.16	7.51
1432	" $\frac{5}{8}$	"	"	"	46.60	1.19	2340	2609	7.09	7.48
1433	" $\frac{11}{16}$	"	"	"	48.85	1.14	2404	2715	7.02	7.46
1434	" $\frac{3}{4}$	"	"	"	51.10	1.09	2467	2819	6.95	7.43
			18″ × 21″ Section.	B Series						
*1435	18x$\frac{3}{8}$	21x$\frac{1}{2}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{3}{8}$	35.06	2.49	1779	1805	7.12	7.18
*1436	" $\frac{7}{16}$	"	"	"	37.31	2.34	1853	1901	7.05	7.14
1437	" $\frac{1}{2}$	"	"	"	39.56	2.21	1925	1996	6.98	7.10
1438	" $\frac{9}{16}$	"	"	"	41.81	2.09	1995	2090	6.91	7.07
1439	" $\frac{5}{8}$	"	"	"	44.06	1.98	2065	2183	6.84	7.04
1440	" $\frac{11}{16}$	"	"	"	46.31	1.89	2135	2275	6.79	7.01
1441	" $\frac{3}{4}$	"	"	"	48.56	1.80	2204	2366	6.74	6.98
*1442	18x$\frac{3}{8}$	21x$\frac{1}{2}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{7}{16}$	36.02	2.21	1883	1880	7.23	7.23
*1443	" $\frac{7}{16}$	"	"	"	38.27	2.08	1954	1977	7.14	7.19
1444	" $\frac{1}{2}$	"	"	"	40.52	1.97	2024	2072	7.06	7.15
1445	" $\frac{9}{16}$	"	"	"	42.77	1.86	2093	2166	6.99	7.12
1446	" $\frac{5}{8}$	"	"	"	45.02	1.77	2161	2259	6.93	7.09
1447	" $\frac{11}{16}$	"	"	"	47.27	1.69	2229	2351	6.87	7.06
1448	" $\frac{3}{4}$	"	"	"	49.52	1.61	2296	2443	6.81	7.03

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

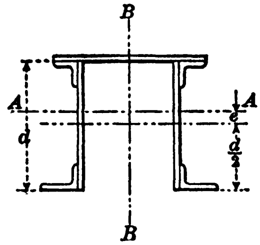
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1449	18x3/8	21x1/2	3½x3½x3/8	5x3½x½	36.96	1.96	1975	1957	7.31	7.28
*1450	" 7/16	"	"	"	39.21	1.84	2045	2053	7.22	7.24
1451	" ½	"	"	"	41.46	1.74	2112	2147	7.14	7.20
1452	" 9/16	"	"	"	43.71	1.65	2180	2242	7.06	7.16
1453	" 5/8	"	"	"	45.96	1.57	2247	2335	6.99	7.13
1454	" 11/16	"	"	"	48.21	1.50	2313	2427	6.93	7.10
1455	" ¾	"	"	"	50.46	1.43	2379	2518	6.87	7.07
*1456	18x3/8	21x½	3½x3½x3/8	5x3½x9/16	37.90	1.71	2066	2033	7.38	7.32
*1457	" 7/16	"	"	"	40.15	1.61	2134	2129	7.29	7.28
1458	" ½	"	"	"	42.40	1.53	2200	2224	7.19	7.24
1459	" 9/16	"	"	"	44.65	1.45	2265	2318	7.12	7.21
1460	" 5/8	"	"	"	46.90	1.38	2331	2411	7.05	7.17
1461	" 11/16	"	"	"	49.15	1.32	2395	2503	6.98	7.14
1462	" ¾	"	"	"	51.40	1.26	2460	2594	6.92	7.10
*1463	18x3/8	21x½	3½x3½x3/8	5x3½x5/8	38.80	1.48	2145	2106	7.44	7.37
*1464	" 7/16	"	"	"	41.05	1.40	2211	2203	7.34	7.33
1465	" ½	"	"	"	43.30	1.33	2276	2298	7.25	7.29
1466	" 9/16	"	"	"	45.55	1.26	2340	2392	7.17	7.25
1467	" 5/8	"	"	"	47.80	1.20	2405	2485	7.09	7.21
1468	" 11/16	"	"	"	50.05	1.15	2439	2577	7.02	7.18
1469	" ¾	"	"	"	52.30	1.10	2532	2668	6.96	7.14
*1470	18x3/8	21x½	3½x3½x3/8	5x3½x11/16	39.70	1.27	2224	2180	7.47	7.41
*1471	" 7/16	"	"	"	41.95	1.20	2288	2276	7.38	7.37
1472	" ½	"	"	"	44.20	1.14	2351	2371	7.29	7.33
1473	" 9/16	"	"	"	46.45	1.09	2415	2465	7.21	7.29
1474	" 5/8	"	"	"	48.70	1.04	2478	2558	7.13	7.25
1475	" 11/16	"	"	"	50.95	0.99	2542	2650	7.06	7.21
1476	" ¾	"	"	"	53.20	0.95	2604	2741	7.00	7.18
*1477	18x3/8	21x½	3½x3½x3/8	5x3½x¾	40.58	1.08	2293	2255	7.51	7.45
*1478	" 7/16	"	"	"	42.83	1.02	2356	2351	7.42	7.41
1479	" ½	"	"	"	45.08	0.97	2419	2446	7.32	7.37
1480	" 9/16	"	"	"	47.33	0.93	2481	2540	7.24	7.33
1481	" 5/8	"	"	"	49.58	0.89	2546	2633	7.16	7.29
1482	" 11/16	"	"	"	51.83	0.85	2607	2725	7.09	7.25
1483	" ¾	"	"	"	54.08	0.81	2670	2816	7.03	7.21

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

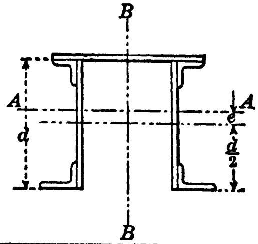
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area.	Eccentricity.	Moments of Inertia. Axis A-A.	Moments of Inertia. Axis B-B.	Radii of Gyration. Axis A-A.	Radii of Gyration. Axis B-B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
				18″ × 22″ Section. A Series.						
*1484	18x$\frac{7}{16}$	22x$\frac{1}{2}$	3x3x$\frac{3}{8}$	4x3x$\frac{3}{8}$	35.93	2.65	1735	2170	6.95	7.77
1485	“ $\frac{1}{2}$	“	“	“	38.18	2.49	1811	2297	6.89	7.76
1486	“ $\frac{9}{16}$	“	“	“	40.43	2.35	1885	2422	6.83	7.74
1487	“ $\frac{5}{8}$	“	“	“	42.68	2.23	1957	2545	6.77	7.72
1488	“ $\frac{11}{16}$	“	“	“	44.93	2.12	2028	2667	6.72	7.70
1489	“ $\frac{3}{4}$	“	“	“	47.18	2.02	2099	2787	6.67	7.68
*1490	18x$\frac{7}{16}$	22x$\frac{1}{2}$	3x3x$\frac{3}{8}$	4x3x$\frac{7}{16}$	36.71	2.42	1823	2240	7.05	7.81
1491	“ $\frac{1}{2}$	“	“	“	38.96	2.28	1896	2367	6.98	7.80
1492	“ $\frac{9}{16}$	“	“	“	41.21	2.16	1968	2492	6.91	7.78
1493	“ $\frac{5}{8}$	“	“	“	43.46	2.05	2038	2615	6.85	7.76
1494	“ $\frac{11}{16}$	“	“	“	45.71	1.94	2108	2737	6.79	7.74
1495	“ $\frac{3}{4}$	“	“	“	47.96	1.85	2177	2857	6.74	7.72
*1496	18x$\frac{7}{16}$	22x$\frac{1}{2}$	3x3x$\frac{3}{8}$	4x3x$\frac{1}{2}$	37.47	2.21	1904	2310	7.13	7.85
1497	“ $\frac{1}{2}$	“	“	“	39.72	2.09	1975	2437	7.05	7.83
1498	“ $\frac{9}{16}$	“	“	“	41.97	1.97	2045	2562	6.98	7.81
1499	“ $\frac{5}{8}$	“	“	“	44.22	1.87	2114	2685	6.92	7.79
1500	“ $\frac{11}{16}$	“	“	“	46.47	1.78	2182	2807	6.85	7.77
1501	“ $\frac{3}{4}$	“	“	“	48.72	1.70	2250	2927	6.80	7.75
*1502	18x$\frac{7}{16}$	22x$\frac{1}{2}$	3x3x$\frac{3}{8}$	4x3x$\frac{9}{16}$	38.21	2.02	1979	2379	7.20	7.89
1503	“ $\frac{1}{2}$	“	“	“	40.46	1.90	2048	2506	7.12	7.87
1504	“ $\frac{9}{16}$	“	“	“	42.71	1.80	2117	2631	7.04	7.85
1505	“ $\frac{5}{8}$	“	“	“	44.96	1.71	2184	2754	6.97	7.83
1506	“ $\frac{11}{16}$	“	“	“	47.21	1.63	2251	2876	6.90	7.80
1507	“ $\frac{3}{4}$	“	“	“	49.46	1.56	2318	2996	6.85	7.78
*1508	18x$\frac{7}{16}$	22x$\frac{1}{2}$	3x3x$\frac{3}{8}$	4x3x$\frac{5}{8}$	38.93	1.83	2049	2445	7.26	7.93
1509	“ $\frac{1}{2}$	“	“	“	41.18	1.73	2118	2572	7.17	7.90
1510	“ $\frac{9}{16}$	“	“	“	43.43	1.64	2185	2697	7.09	7.88
1511	“ $\frac{5}{8}$	“	“	“	45.68	1.56	2251	2820	7.02	7.86
1512	“ $\frac{11}{16}$	“	“	“	47.93	1.49	2317	2942	6.95	7.84
1513	“ $\frac{3}{4}$	“	“	“	50.18	1.42	2383	3062	6.89	7.81

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



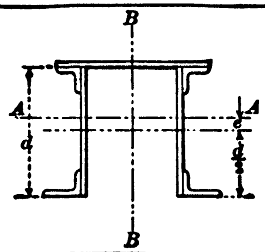
Four Angles and Three Plates.

Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A–A. I_A	Moments of Inertia. Axis B–B. I_B	Radii of Gyration. Axis A–A. r_A	Radii of Gyration. Axis B–B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1514	18x7/16	22x1/2	3x3x3/8	4x3x11/16	39.65	1.65	2116	2513	7.30	7.96
1515	“ 1/2	“	“	“	41.90	1.57	2183	2640	7.22	7.94
1516	“ 9/16	“	“	“	44.15	1.49	2249	2765	7.14	7.92
1517	“ 5/8	“	“	“	46.40	1.41	2314	2888	7.06	7.89
1518	“ 11/16	“	“	“	48.65	1.34	2379	3010	6.99	7.87
1519	“ 3/4	“	“	“	50.90	1.29	2444	3130	6.93	7.84
*1520	18x7/16	22x1/2	3x3x3/8	4x3x3/4	40.35	1.49	2177	2581	7.35	8.00
1521	“ 1/2	“	“	“	42.60	1.41	2243	2708	7.26	7.97
1522	“ 9/16	“	“	“	44.85	1.34	2308	2833	7.17	7.95
1523	“ 5/8	“	“	“	47.10	1.28	2372	2956	7.09	7.92
1524	“ 11/16	“	“	“	49.35	1.22	2437	3078	7.03	7.90
1525	“ 3/4	“	“	“	51.60	1.17	2501	3198	6.96	7.87
			18″ × 22″ Section.	B Series.						
*1526	18x3/8	22x1/2	3½x3½x3/8	5x3½x3/8	35.56	2.59	1801	2052	7.11	7.60
*1527	“ 7/16	“	“	“	37.81	2.43	1877	2166	7.05	7.57
1528	“ 1/2	“	“	“	40.06	2.30	1950	2277	6.98	7.54
1529	“ 9/16	“	“	“	42.31	2.17	2021	2386	6.92	7.51
1530	“ 5/8	“	“	“	44.56	2.06	2093	2493	6.86	7.48
1531	“ 11/16	“	“	“	46.81	1.96	2163	2599	6.80	7.45
1532	“ 3/4	“	“	“	49.06	1.87	2232	2702	6.75	7.42
*1533	18x3/8	22x1/2	3½x3½x3/8	5x3½x7/16	36.52	2.31	1906	2137	7.23	7.65
*1534	“ 7/16	“	“	“	38.77	2.18	1978	2250	7.14	7.62
1535	“ 1/2	“	“	“	41.02	2.06	2049	2361	7.07	7.59
1536	“ 9/16	“	“	“	43.27	1.95	2118	2470	7.00	7.56
1537	“ 5/8	“	“	“	45.52	1.85	2188	2577	6.93	7.53
1538	“ 11/16	“	“	“	47.77	1.76	2257	2683	6.87	7.50
1539	“ 3/4	“	“	“	50.02	1.68	2324	2787	6.82	7.47
*1540	18x3/8	22x1/2	3½x3½x3/8	5x3½x1/2	37.46	2.05	2002	2222	7.31	7.70
*1541	“ 7/16	“	“	“	39.71	1.93	2072	2335	7.22	7.67
1542	“ 1/2	“	“	“	41.96	1.83	2141	2446	7.14	7.64
1543	“ 9/16	“	“	“	44.21	1.74	2208	2555	7.06	7.60
1544	“ 5/8	“	“	“	46.46	1.65	2276	2662	7.00	7.57
1545	“ 11/16	“	“	“	48.71	1.58	2343	2768	6.94	7.54
1546	“ 3/4	“	“	“	50.96	1.15	2409	2872	6.88	7.51

* Spacing of rivet lines of web greater than 30 × thickness of plate.

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

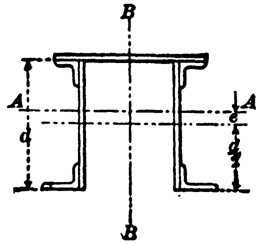
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1547	18x$\frac{3}{8}$	22x$\frac{1}{2}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{9}{16}$	38.40	1.81	2093	2306	7.38	7.75
*1548	" $\frac{7}{16}$	"	"	"	40.65	1.71	2161	2419	7.29	7.71
1549	" $\frac{1}{2}$	"	"	"	42.90	1.62	2229	2530	7.21	7.68
1550	" $\frac{9}{16}$	"	"	"	45.15	1.54	2294	2639	7.13	7.64
1551	" $\frac{5}{8}$	"	"	"	47.40	1.47	2360	2746	7.06	7.61
1552	" $\frac{11}{16}$	"	"	"	49.65	1.40	2426	2852	6.99	7.58
1553	" $\frac{3}{4}$	"	"	"	51.90	1.34	2491	2956	6.93	7.54
*1554	18x$\frac{3}{8}$	22x$\frac{1}{2}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{5}{8}$	39.30	1.58	2177	2388	7.44	7.80
*1555	" $\frac{7}{16}$	"	"	"	41.55	1.50	2243	2502	7.35	7.76
1556	" $\frac{1}{2}$	"	"	"	43.80	1.42	2309	2613	7.26	7.73
1557	" $\frac{9}{16}$	"	"	"	46.05	1.35	2373	2722	7.18	7.69
1558	" $\frac{5}{8}$	"	"	"	48.30	1.29	2438	2829	7.11	7.66
1559	" $\frac{11}{16}$	"	"	"	50.55	1.23	2502	2935	7.04	7.62
1560	" $\frac{3}{4}$	"	"	"	52.80	1.18	2566	3039	6.97	7.59
*1561	18x$\frac{3}{8}$	22x$\frac{1}{2}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{11}{16}$	40.20	1.37	2255	2470	7.49	7.84
*1562	" $\frac{7}{16}$	"	"	"	42.45	1.30	2320	2584	7.39	7.80
1563	" $\frac{1}{2}$	"	"	"	44.70	1.24	2385	2695	7.30	7.77
1564	" $\frac{9}{16}$	"	"	"	46.95	1.18	2448	2804	7.22	7.73
1565	" $\frac{5}{8}$	"	"	"	49.20	1.12	2512	2911	7.15	7.69
1566	" $\frac{11}{16}$	"	"	"	51.45	1.07	2576	3017	7.08	7.66
1567	" $\frac{3}{4}$	"	"	"	53.70	1.03	2639	3121	7.01	7.63
*1568	18x$\frac{3}{8}$	22x$\frac{1}{2}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{3}{4}$	41.08	1.18	2326	2553	7.53	7.89
*1569	" $\frac{7}{16}$	"	"	"	43.33	1.12	2390	2667	7.43	7.85
1570	" $\frac{1}{2}$	"	"	"	45.58	1.06	2454	2778	7.34	7.81
1571	" $\frac{9}{16}$	"	"	"	47.83	1.01	2516	2887	7.25	7.77
1572	" $\frac{5}{8}$	"	"	"	50.08	0.97	2579	2994	7.17	7.73
1573	" $\frac{11}{16}$	"	"	"	52.33	0.93	2642	3100	7.11	7.70
1574	" $\frac{3}{4}$	"	"	"	54.58	0.89	2705	3204	7.04	7.66
20″ × 23″ Section. A Series.										
*1575	20x$\frac{1}{2}$	23x$\frac{1}{2}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	5x3$\frac{1}{2}$x$\frac{3}{8}$	42.56	2.51	2530	2697	7.71	7.97
1576	" $\frac{9}{16}$	"	"	"	45.06	2.37	2628	2836	7.64	7.94
1577	" $\frac{5}{8}$	"	"	"	47.56	2.25	2724	2973	7.57	7.91
1578	" $\frac{11}{16}$	"	"	"	50.06	2.13	2820	3107	7.51	7.88
1579	" $\frac{3}{4}$	"	"	"	52.56	2.03	2914	3239	7.45	7.85

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

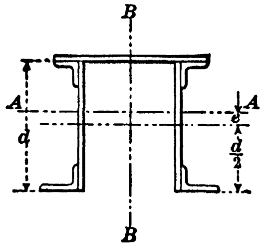
Section Number.	Plates.		Angles.		Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.			Axis. A–A.	Axis B–B.	Axis A–A.	Axis B–B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1580	20x½	23x½	3½x3½x⅜	5x3½x7/16	43.52	2.25	2655	2790	7.81	8.01
1581	" 9/16	"	"	"	46.02	2.13	2750	2929	7.73	7.98
1582	" ⅝	"	"	"	48.52	2.02	2844	3066	7.66	7.95
1583	" 11/16	"	"	"	51.02	1.92	2938	3200	7.59	7.92
1584	" ¾	"	"	"	53.52	1.83	3029	3332	7.52	7.89
*1585	20x½	23x½	3½x3½x⅜	5x3½x½	44.46	2.02	2769	2884	7.89	8.06
1586	" 9/16	"	"	"	46.96	1.91	2862	3023	7.81	8.03
1587	" ⅝	"	"	"	49.46	1.82	2954	3160	7.73	8.00
1588	" 11/16	"	"	"	51.96	1.73	3046	3294	7.66	7.96
1589	" ¾	"	"	"	54.46	1.65	3136	3426	7.59	7.93
*1590	20x½	23x½	3½x3½x⅜	5x3½x9/16	45.40	1.79	2880	2978	7.97	8.10
1591	" 9/16	"	"	"	47.90	1.70	2971	3117	7.89	8.07
1592	" ⅝	"	"	"	50.40	1.62	3061	3254	7.80	8.04
1593	" 11/16	"	"	"	52.90	1.54	3151	3388	7.72	8.00
1594	" ¾	"	"	"	55.40	1.47	3239	3520	7.64	7.97
*1595	20x½	23x½	3½x3½x⅜	5x3½x⅝	46.30	1.59	2980	3068	8.03	8.14
1596	" 9/16	"	"	"	48.80	1.50	3069	3207	7.93	8.11
1597	" ⅝	"	"	"	51.30	1.43	3158	3344	7.85	8.07
1598	" 11/16	"	"	"	53.80	1.36	3247	3478	7.77	8.04
1599	" ¾	"	"	"	56.30	1.30	3334	3610	7.70	8.01
*1600	20x½	23x½	3½x3½x⅜	5x3½x11/16	47.20	1.39	3077	3159	8.08	8.18
1601	" 9/16	"	"	"	49.70	1.32	3164	3298	7.98	8.14
1602	" ⅝	"	"	"	52.20	1.26	3251	3435	7.90	8.11
1603	" 11/16	"	"	"	54.70	1.20	3339	3569	7.82	8.08
1604	" ¾	"	"	"	57.20	1.15	3426	3701	7.74	8.05
*1605	20x½	23x½	3½x3½x⅜	5x3½x¾	48.08	1.21	3164	3251	8.11	8.23
1606	" 9/16	"	"	"	50.58	1.15	3250	3390	8.02	8.19
1607	" ⅝	"	"	"	53.08	1.09	3336	3527	7.93	8.15
1608	" 11/16	"	"	"	55.58	1.04	3423	3661	7.85	8.12
1609	" ¾	"	"	"	58.08	1.00	3509	3793	7.77	8.08

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



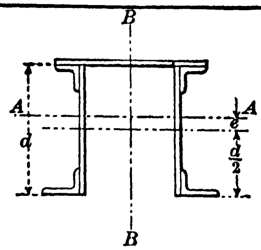
Four Angles and Three Plates.

Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
					20" × 23" Section. B Series.					
*1610	20x7/16	23x1/2	4x4x7/16	6x4x7/16	43.98	2.29	2782	2721	7.95	7.86
1611	" 1/2	"	"	"	46.48	2.17	2877	2845	7.87	7.82
1612	" 9/16	"	"	"	48.98	2.06	2973	2966	7.79	7.78
1613	" 5/8	"	"	"	51.48	1.96	3066	3085	7.72	7.74
1614	" 11/16	"	"	"	53.98	1.87	3158	3202	7.65	7.70
1615	" 3/4	"	"	"	56.48	1.78	3250	3317	7.58	7.66
*1616	20x7/16	23x1/2	4x4x7/16	6x4x1/2	45.12	2.01	2919	2832	8.04	7.92
1617	" 1/2	"	"	"	47.62	1.91	3012	2956	7.95	7.88
1618	" 9/16	"	"	"	50.12	1.81	3104	3077	7.87	7.84
1619	" 5/8	"	"	"	52.62	1.73	3195	3196	7.79	7.79
1620	" 11/16	"	"	"	55.12	1.65	3285	3313	7.72	7.75
1621	" 3/4	"	"	"	57.62	1.58	3376	3428	7.65	7.71
*1622	20x7/16	23x1/2	4x4x7/16	6x4x9/16	46.24	1.75	3050	2941	8.12	7.97
1623	" 1/2	"	"	"	48.74	1.66	3140	3065	8.03	7.93
1624	" 9/16	"	"	"	51.24	1.58	3230	3186	7.94	7.88
1625	" 5/8	"	"	"	53.74	1.51	3319	3305	7.86	7.84
1626	" 11/16	"	"	"	56.24	1.44	3408	3422	7.78	7.80
1627	" 3/4	"	"	"	58.74	1.38	3497	3537	7.72	7.76
*1628	20x7/16	23x1/2	4x4x7/16	6x4x5/8	47.34	1.51	3170	3048	8.18	8.02
1629	" 1/2	"	"	"	49.84	1.43	3258	3172	8.08	7.98
1630	" 9/16	"	"	"	52.34	1.36	3347	3293	8.00	7.93
1631	" 5/8	"	"	"	54.84	1.30	3434	3412	7.92	7.89
1632	" 11/16	"	"	"	57.34	1.24	3521	3529	7.84	7.84
1633	" 3/4	"	"	"	59.84	1.19	3609	3644	7.77	7.80
*1634	20x7/16	23x1/2	4x4x7/16	6x4x11/16	48.42	1.28	3279	3157	8.23	8.08
1635	" 1/2	"	"	"	50.92	1.22	3366	3281	8.13	8.03
1636	" 9/16	"	"	"	53.42	1.16	3453	3402	8.04	7.98
1637	" 5/8	"	"	"	55.92	1.11	3539	3521	7.96	7.94
1638	" 11/16	"	"	"	58.42	1.06	3625	3638	7.88	7.89
1639	" 3/4	"	"	"	60.92	1.02	3712	3753	7.81	7.85

* Spacing of rivet lines of web greater than 30 × thickness of plate.

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Four Angles and Three Plates.

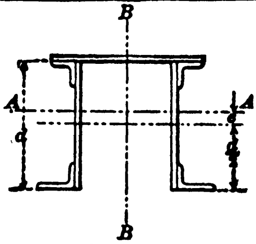
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1640	20x7/16	23x1/2	4x4x7/16	6x4x3/4	49.50	1.06	3384	3265	8.27	8.12
1641	" 1/2	"	"	"	52.00	1.01	3470	3389	8.17	8.07
1642	" 9/16	"	"	"	54.50	0.96	3556	3510	8.08	8.02
1643	" 5/8	"	"	"	57.00	0.92	3641	3629	7.99	7.98
1644	" 11/16	"	"	"	59.50	0.88	3726	3746	7.91	7.93
1645	" 3/4	"	"	"	62.00	0.85	3812	3861	7.84	7.89
20″ × 24″ Section. A Series.										
*1646	20x1/2	24x9/16	3½x3½x3/8	5x3½x3/8	44.56	2.87	2651	3104	7.71	8.35
1647	" 9/16	"	"	"	47.06	2.71	2754	3262	7.65	8.33
1648	" 5/8	"	"	"	49.56	2.57	2855	3418	7.59	8.31
1649	" 11/16	"	"	"	52.06	2.45	2954	3572	7.54	8.29
1650	" 3/4	"	"	"	54.56	2.34	3051	3724	7.48	8.27
*1651	20x1/2	24x9/16	3½x3½x3/8	5x3½x7/16	45.52	2.61	2784	3207	7.82	8.39
1652	" 9/16	"	"	"	48.02	2.48	2883	3365	7.75	8.37
1653	" 5/8	"	"	"	50.52	2.36	2980	3521	7.68	8.34
1654	" 11/16	"	"	"	53.02	2.25	3077	3675	7.62	8.32
1655	" 3/4	"	"	"	55.52	2.14	3173	3827	7.56	8.30
*1656	20x1/2	24x9/16	3½x3½x3/8	5x3½x1/2	46.46	2.38	2907	3310	7.91	8.44
1657	" 9/16	"	"	"	48.96	2.26	3003	3468	7.83	8.41
1658	" 5/8	"	"	"	51.46	2.15	3098	3624	7.76	8.39
1659	" 11/16	"	"	"	53.96	2.05	3193	3778	7.69	8.37
1660	" 3/4	"	"	"	56.46	1.96	3286	3930	7.63	8.34
*1661	20x1/2	24x9/16	3½x3½x3/8	5x3½x9/16	47.40	2.16	3024	3413	7.98	8.49
1662	" 9/16	"	"	"	49.90	2.05	3118	3571	7.90	8.46
1663	" 5/8	"	"	"	52.40	1.95	3211	3727	7.83	8.44
1664	" 11/16	"	"	"	54.90	1.86	3305	3881	7.76	8.41
1665	" 3/4	"	"	"	57.40	1.78	3396	4033	7.69	8.38
*1666	20x1/2	24x9/16	3½x3½x3/8	5x3½x5/8	48.30	1.95	3132	3513	8.05	8.53
1667	" 9/16	"	"	"	50.80	1.86	3224	3671	7.97	8.50
1668	" 5/8	"	"	"	53.30	1.77	3315	3827	7.89	8.47
1669	" 11/16	"	"	"	55.80	1.69	3407	3981	7.81	8.45
1670	" 3/4	"	"	"	58.30	1.62	3497	4133	7.74	8.42

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



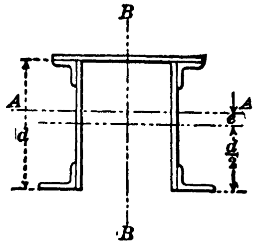
Four Angles and Three Plates.

Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1671	20x½	24x9/16	3½x3½x⅜	5x3½x11/16	49.20	1.76	3234	3613	8.11	8.57
1672	" 9/16	"	"	"	51.70	1.67	3325	3771	8.02	8.54
1673	" ⅝	"	"	"	54.20	1.60	3414	3927	7.94	8.51
1674	" 11/16	"	"	"	56.70	1.53	3504	4081	7.86	8.48
1675	" ¾	"	"	"	59.20	1.46	3593	4233	7.79	8.45
*1676	20x½	24x9/16	3½x3½x⅜	5x3½x¾	50.08	1.57	3329	3714	8.15	8.61
1677	" 9/16	"	"	"	52.58	1.50	3418	3872	8.06	8.58
1678	" ⅝	"	"	"	55.08	1.43	3506	4028	7.98	8.55
1679	" 11/16	"	"	"	57.58	1.37	3595	4182	7.90	8.52
1680	" ¾	"	"	"	60.08	1.31	3683	4334	7.83	8.49
				20″ × 24″ Section.	B Series.					
*1681	20x7/16	24x9/16	4x4x7/16	6x4x7/16	45.98	2.65	2910	3134	7.95	8.26
1682	" ½	"	"	"	48.48	2.51	3009	3276	7.88	8.22
1683	" 9/16	"	"	"	50.98	2.39	3108	3415	7.81	8.18
1684	" ⅝	"	"	"	53.48	2.28	3205	3552	7.74	8.15
1685	" 11/16	"	"	"	55.98	2.17	3300	3687	7.68	8.11
1686	" ¾	"	"	"	58.48	2.08	3396	3820	7.62	8.08
*1687	20x7/16	24x9/16	4x4x7/16	6x4x½	47.12	2.37	3056	3257	8.05	8.31
1688	" ½	"	"	"	49.62	2.25	3152	3399	7.97	8.28
1689	" 9/16	"	"	"	52.12	2.14	3248	3538	7.90	8.24
1690	" ⅝	"	"	"	54.62	2.05	3343	3675	7.82	8.20
1691	" 11/16	"	"	"	57.12	1.96	3435	3810	7.76	8.17
1692	" ¾	"	"	"	59.62	1.87	3528	3943	7.69	8.13
*1693	20x7/16	24x9/16	4x4x7/16	6x4x9/16	48.24	2.11	3194	3375	8.14	8.37
1694	" ½	"	"	"	50.74	2.01	3288	3517	8.05	8.33
1695	" 9/16	"	"	"	53.24	1.91	3381	3656	7.97	8.29
1696	" ⅝	"	"	"	55.74	1.83	3473	3793	7.89	8.25
1697	" 11/16	"	"	"	58.24	1.75	3564	3928	7.82	8.21
1698	" ¾	"	"	"	60.74	1.68	3655	4061	7.76	8.17
*1699	20x7/16	24x9/16	4x4x7/16	6x4x⅝	49.34	1.87	3323	3495	8.21	8.41
1700	" ½	"	"	"	51.84	1.78	3414	3637	8.12	8.38
1701	" 9/16	"	"	"	54.34	1.70	3506	3776	8.03	8.34
1702	" ⅝	"	"	"	56.84	1.62	3595	3913	7.95	8.30
1703	" 11/16	"	"	"	59.34	1.55	3685	4048	7.88	8.26
1704	" ¾	"	"	"	61.84	1.49	3775	4181	7.81	8.22

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*
PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

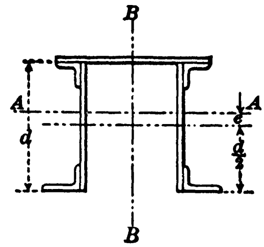
Section Number.	Plates.		Angles.		Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.			Axis A-A.	Axis B-B.	Axis A-A.	Axis B-B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1705	20x7/16	24x9/16	4x4x7/16	6x4x11/16	50.42	1.64	3441	3615	8.26	8.47
1706	" 1/2	"	"	"	52.92	1.57	3530	3757	8.17	8.43
1707	" 9/16	"	"	"	55.42	1.50	3620	3896	8.08	8.39
1708	" 5/8	"	"	"	57.92	1.43	3708	4033	8.00	8.35
1709	" 11/16	"	"	"	60.42	1.37	3796	4168	7.93	8.31
1710	" 3/4	"	"	"	62.92	1.32	3885	4301	7.86	8.27
*1711	20x7/16	24x9/16	4x4x7/16	6x4x3/4	51.50	1.43	3554	3733	8.31	8.51
1712	" 1/2	"	"	"	54.00	1.36	3642	3875	8.21	8.47
1713	" 9/16	"	"	"	56.50	1.30	3730	4014	8.12	8.43
1714	" 5/8	"	"	"	59 00	1.25	3817	4151	8.04	8.39
1715	" 11/16	"	"	"	61.50	1.20	3904	4286	7.97	8.35
1716	" 3/4	"	"	"	64.00	1.15	3992	4419	7.90	8.31
22″ × 25″ Section. A Series.										
*1717	22x9/16	25x9/16	3½x3½x7/16	5x3½x½	52.55	2.57	3839	4129	8.55	8.87
1718	" 5/8	"	"	"	55.30	2.44	3967	4323	8.47	8.84
1719	" 11/16	"	"	"	58.05	2.33	4093	4514	8.40	8.82
1720	" 3/4	"	"	"	60.80	2.22	4219	4703	8.33	8.80
*1721	22x9/16	25x9/16	3½x3½x7/16	5x3½x9/16	53.49	2.35	3983	4242	8.63	8.90
1722	" 5/8	"	"	"	56.24	2.24	4108	4436	8.54	8.88
1723	" 11/16	"	"	"	58.99	2.14	4232	4627	8.47	8.86
1724	" 3/4	"	"	"	61.74	2.04	4355	4816	8.40	8.83
*1725	22x9/16	25x9/16	3½x3½x7/16	5x3½x5/8	54.39	2.15	4116	4350	8.70	8.94
1726	" 5/8	"	"	"	57.14	2.05	4238	4544	8.61	8.92
1727	" 11/16	"	"	"	59.89	1.95	4361	4735	8.53	8.89
1728	" 3/4	"	"	"	62.64	1.86	4483	4924	8.46	8.87
*1729	22x9/16	25x9/16	3½x3½x7/16	5x3½x11/16	55.29	1.96	4242	4460	8.76	8.98
1730	" 5/8	"	"	"	58.04	1.86	4363	4654	8.67	8.96
1731	" 11/16	"	"	"	60.79	1.78	4483	4845	8.59	8.93
1732	" 3/4	"	"	"	63.54	1.70	4603	5034	8.51	8.90

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Four Angles and Three Plates.

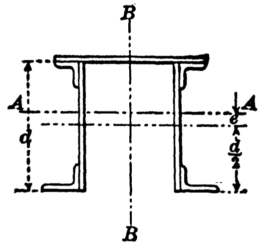
Section Number.	Plates.		Angles.		Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.			Axis A-A.	Axis B-B.	Axis A-A.	Axis B-B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1733	22x$\frac{9}{16}$	25x$\frac{9}{16}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{7}{16}$	5x3$\frac{1}{2}$x$\frac{3}{4}$	56.17	1.77	4361	4570	8.81	9.02
1734	" $\frac{5}{8}$	"	"	"	58.92	1.69	4480	4764	8.72	8.99
1735	" $\frac{11}{16}$	"	"	"	61.67	1.62	4598	4955	8.63	8.96
1736	" $\frac{3}{4}$	"	"	"	64.42	1.55	4716	5144	8.55	8.93
22″ X 25″ Section. B Series.										
*1737	22x$\frac{1}{2}$	25x$\frac{9}{16}$	4x4x$\frac{7}{16}$	6x4x$\frac{1}{2}$	52.18	2.47	3974	3939	8.73	8.69
*1738	" $\frac{9}{16}$	"	"	"	54.93	2.34	4102	4113	8.64	8.65
1739	" $\frac{5}{8}$	"	"	"	57.68	2.23	4227	4284	8.56	8.62
1740	" $\frac{11}{16}$	"	"	"	60.43	2.13	4351	4453	8.49	8.58
1741	" $\frac{3}{4}$	"	"	"	63.18	2.04	4473	4620	8.41	8.55
*1742	22x$\frac{1}{2}$	25x$\frac{9}{16}$	4x4x$\frac{7}{16}$	6x4x$\frac{9}{16}$	53.30	2.21	4141	4070	8.81	8.74
*1743	" $\frac{9}{16}$	"	"	"	56.05	2.10	4265	4244	8.72	8.70
1744	" $\frac{5}{8}$	"	"	"	58.80	2.00	4388	4415	8.64	8.67
1745	" $\frac{11}{16}$	"	"	"	61 55	1.91	4509	4584	8.56	8.63
1746	" $\frac{3}{4}$	"	"	"	64.30	1.83	4630	4751	8.49	8.60
*1747	22x$\frac{1}{2}$	25x$\frac{9}{16}$	4x4x$\frac{7}{16}$	6x4x$\frac{5}{8}$	54.40	1.96	4299	4200	8.89	8.79
*1748	" $\frac{9}{16}$	"	"	"	57.15	1.87	4419	4374	8.79	8.75
1749	" $\frac{5}{8}$	"	"	"	59.90	1.78	4539	4545	8.70	8.71
1750	" $\frac{11}{16}$	"	"	"	62.65	1.70	4659	4714	8.62	8.67
1751	" $\frac{3}{4}$	"	"	"	65.40	1.63	4778	4881	8.54	8.64
*1752	22x$\frac{1}{2}$	25x$\frac{9}{16}$	4x4x$\frac{7}{16}$	6x4x$\frac{11}{16}$	55.48	1.74	4441	4331	8.95	8.84
*1753	" $\frac{9}{16}$	"	"	"	58.23	1.66	4560	4505	8.85	8.80
1754	" $\frac{5}{8}$	"	"	"	60.98	1.58	4678	4676	8.76	8.76
1755	" $\frac{11}{16}$	"	"	"	63.73	1.51	4796	4845	8.68	8.72
1756	" $\frac{3}{4}$	"	"	"	66.48	1.45	4913	5012	8.60	8.68
*1757	22x$\frac{1}{2}$	25x$\frac{9}{16}$	4x4x$\frac{7}{16}$	6x4x$\frac{3}{4}$	56.56	1.52	4580	4461	9.00	8.88
*1758	" $\frac{9}{16}$	"	"	"	59.31	1.45	4697	4635	8.90	8.84
1759	" $\frac{5}{8}$	"	"	"	62.06	1.39	4814	4806	8.81	8.80
1760	" $\frac{11}{16}$	"	"	"	64.81	1.33	4930	4975	8.72	8.76
1761	" $\frac{3}{4}$	"	"	"	67.56	1.27	5046	5142	8.64	8.73

* Spacing of rivet lines of web greater than 30 X thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

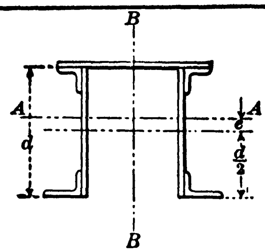
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
				22″ × 26″ Section. A Series.						
*1762	22x9/16	26x5/8	3½x3½x7/16	5x3½x½	54.74	2.93	4006	4681	8.56	9.25
1763	" 5/8	"	"	"	57.49	2.80	4138	4901	8.48	9.23
1764	" 11/16	"	"	"	60.24	2.67	4270	5116	8.41	9.21
1765	" 3/4	"	"	"	62.99	2.54	4402	5326	8.36	9.19
*1766	22x9/16	26x5/8	3½x3½x7/16	5x3½x9/16	55.68	2.71	4160	4804	8.64	9.29
1767	" 5/8	"	"	"	58.43	2.59	4289	5024	8.57	9.27
1768	" 11/16	"	"	"	61.18	2.47	4418	5239	8.50	9.25
1769	" 3/4	"	"	"	63.93	2.36	4546	5449	8.43	9.23
*1770	22x9/16	26x5/8	3½x3½x7/16	5x3½x5/8	56.58	2.51	4300	4923	8.72	9.33
1771	" 5/8	"	"	"	59.33	2.40	4427	5143	8.64	9.31
1772	" 11/16	"	"	"	62.08	2.29	4554	5358	8.57	9.29
1773	" 3/4	"	"	"	64.83	2.19	4679	5568	8.50	9.27
*1774	22x9/16	26x5/8	3½x3½x7/16	5x3½x11/16	57.48	2.32	4436	5042	8.78	9.37
1775	" 5/8	"	"	"	60.23	2.21	4562	5262	8.70	9.35
1776	" 11/16	"	"	"	62.98	2.11	4686	5477	8.63	9.33
1777	" 3/4	"	"	"	65.73	2.02	4809	5687	8.56	9.31
*1778	22x9/16	26x5/8	3½x3½x7/16	5x3½x¾	58.36	2.14	4560	5163	8.84	9.41
1779	" 5/8	"	"	"	61.11	2.04	4684	5383	8.76	9.39
1780	" 11/16	"	"	"	63.86	1.95	4806	5598	8.68	9.36
1781	" 3/4	"	"	"	66.61	1.87	4927	5808	8.60	9.34
				22″ × 26″ Section. B Series.						
*1782	22x½	26x5/8	4x4x7/16	6x4x½	54.37	2.83	4148	4475	8.73	9.07
*1783	" 9/16	"	"	"	57.12	2.69	4280	4672	8.65	9.04
1784	" 5/8	"	"	"	59.87	2.57	4410	4866	8.57	9.01
1785	" 11/16	"	"	"	62.62	2.46	4538	5058	8.51	8.99
1786	" 3/4	"	"	"	65.37	2.36	4664	5247	8.45	8.96
*1787	22x½	26x5/8	4x4x7/16	6x4x9/16	55.49	2.57	4325	4619	8.82	9.12
*1788	" 9/16	"	"	"	58.24	2.45	4453	4816	8.74	9.09
1789	" 5/8	"	"	"	60.99	2.34	4580	5010	8.66	9.06
1790	" 11/16	"	"	"	63.74	2.24	4705	5202	8.59	9.03
1791	" 3/4	"	"	"	66.49	2.15	4829	5391	8.52	9.00

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Four Angles and Three Plates.

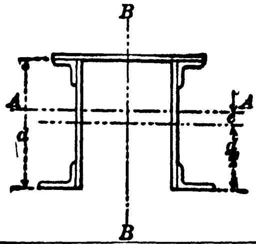
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1792	22x1/2	26x5/8	4x4x7/16	6x4x5/8	56.59	2.33	4490	4761	8.91	9.17
*1793	" 9/16	"	"	"	59.34	2.23	4614	4958	8.82	9.14
1794	" 5/8	"	"	"	62.09	2.13	4738	5152	8.74	9.11
1795	" 11/16	"	"	"	64.84	2.04	4861	5344	8.66	9.08
1796	" 3/4	"	"	"	67.59	1.95	4984	5533	8 59	9.05
*1797	22x1/2	26x5/8	4x4x7/16	6x4x11/16	57.67	2.11	4642	4904	8.97	9.22
*1798	" 9/16	"	"	"	60.42	2.02	4764	5101	8.88	9.19
1799	" 5/8	"	"	"	63.17	1.93	4886	5295	8.80	9.16
1800	" 11/16	"	"	"	65.92	1.85	5007	5487	8.72	9.13
1801	" 3/4	"	"	"	68.67	1.77	5128	5676	8.64	9.09
*1802	22x1/2	26x5/8	4x4x7/16	6x4x3/4	58.75	1.90	4790	5046	9.03	9.27
*1803	" 9/16	"	"	"	61.50	1.81	4911	5243	8.94	9.24
1804	" 5/8	"	"	"	64.25	1.73	5031	5437	8.85	9.20
1805	" 11/16	"	"	"	67.00	1.66	5150	5629	8.77	9.17
1806	" 3/4	"	"	"	69.75	1.60	5268	5818	8.69	9.13
				22" X 28" Section.						
*1807	22x9/16	28x5/8	4x4x3/8	6x4x1/2	57.47	2.77	4326	5601	8.67	9.87
1808	" 5/8	"	"	"	60.22	2.65	4457	5844	8.60	9.85
1809	" 11/16	"	"	"	62.97	2.53	4586	6083	8.53	9.83
1810	" 3/4	"	"	"	65.72	2.42	4714	6320	8.47	9.81
*1811	22x9/16	28x5/8	4x4x3/8	6x4x9/16	58.59	2.53	4502	5771	8.76	9.92
1812	" 5/8	"	"	"	61.34	2.42	4630	6014	8.68	9.90
1813	" 11/16	"	"	"	64.09	2.31	4756	6253	8.61	9.88
1814	" 3/4	"	"	"	66.84	2.22	4881	6490	8.55	9.86
*1815	22x9/16	28x5/8	4x4x3/8	6x4x5/8	59.69	2.30	4666	5939	8.84	9.97
1816	" 5/8	"	"	"	62.44	2.20	4791	6182	8.76	9.95
1817	" 11/16	"	"	"	65.19	2.10	4916	6421	8.68	9.93
1818	" 3/4	"	"	"	67 94	2.02	5038	6658	8.61	9.90
*1819	22x9/16	28x5/8	4x4x3/8	6x4x11/16	60.77	2.09	4818	6108	8.90	10.03
1820	" 5/8	"	"	"	63.52	2.00	4940	6351	8.82	10.00
1821	" 11/16	"	"	"	66.27	1.92	5062	6590	8.74	9.97
1822	" 3/4	"	"	"	69.02	1.84	5182	6827	8.67	9.95

* Spacing of rivet lines of web greater than 30 X thickness of plate.

TABLE 84.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Four Angles and Three Plates

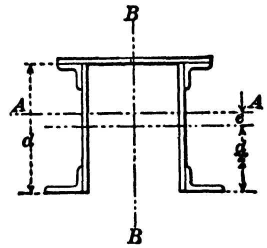
Section Number.	Plates.		Angles.		Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top	Bottom.			Axis A-A.	Axis B-B.	Axis A-A.	Axis B-B.
					A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1823	22x$\frac{9}{16}$	28x$\frac{5}{8}$	4x4x$\frac{3}{8}$	6x4x$\frac{3}{4}$	61.85	1.89	4966	6275	8.96	10.07
1824	" $\frac{5}{8}$	"	"	"	64.60	1.81	5086	6518	8.87	10.04
1825	" $\frac{11}{16}$	"	"	"	67.35	1.73	5206	6757	8.79	10.01
1826	" $\frac{3}{4}$	"	"	"	70.10	1.67	5325	6994	8.72	9.99
24″ X 27″ Section. A Series.										
*1827	24x$\frac{5}{8}$	27x$\frac{5}{8}$	3½x3½x$\frac{7}{16}$	5x3½x½	60.62	3.00	5138	5655	9.21	9.66
1828	" $\frac{11}{16}$	"	"	"	63.62	2.86	5308	5919	9.13	9.64
1829	" $\frac{3}{4}$	"	"	"	66.62	2.73	5476	6174	9.07	9.62
*1830	24x$\frac{5}{8}$	27x$\frac{5}{8}$	3½x3½x$\frac{7}{16}$	5x3½x$\frac{9}{16}$	61.56	2.79	5318	5789	9.29	9.70
1831	" $\frac{11}{16}$	"	"	"	64.36	2.66	5484	6051	9.22	9.68
1832	" $\frac{3}{4}$	"	"	"	67.36	2.54	5648	6308	9.15	9.66
*1833	24x$\frac{5}{8}$	27x$\frac{5}{8}$	3½x3½x$\frac{7}{16}$	5x3½x$\frac{5}{8}$	62.46	2.60	5483	5918	9.37	9.74
1834	" $\frac{11}{16}$	"	"	"	65.46	2.48	5647	6179	9.29	9.72
1835	" $\frac{3}{4}$	"	"	"	68.46	2.37	5809	6437	9.21	9.70
*1836	24x$\frac{5}{8}$	27x$\frac{5}{8}$	3½x3½x$\frac{7}{16}$	5x3½x$\frac{11}{16}$	63.36	2.41	5644	6048	9.44	9.77
1837	" $\frac{11}{16}$	"	"	"	66.36	2.30	5804	6309	9.36	9.75
1838	" $\frac{3}{4}$	"	"	"	69.36	2.20	5964	6567	9.28	9.73
*1839	24x$\frac{5}{8}$	27x$\frac{5}{8}$	3½x3½x$\frac{7}{16}$	5x3½x$\frac{3}{4}$	64.24	2.23	5792	6179	9.49	9.81
1840	" $\frac{11}{16}$	"	"	"	67.24	2.13	5950	6440	9.40	9.79
1841	" $\frac{3}{4}$	"	"	"	70.24	2.04	6107	6698	9.32	9.77
24″ X 27″ Section. B Series.										
*1842	24x$\frac{9}{16}$	27x$\frac{5}{8}$	4x4x$\frac{7}{16}$	6x4x½	60.00	2.92	5296	5372	9.39	9.46
*1843	" $\frac{5}{8}$	"	"	"	63.00	2.78	5464	5610	9.31	9.43
1844	" $\frac{11}{16}$	"	"	"	66.00	2.65	5631	5844	9.24	9.41
1845	" $\frac{3}{4}$	"	"	"	69.00	2.54	5797	6075	9.17	9.39
*1846	24x$\frac{9}{16}$	27x$\frac{5}{8}$	4x4x$\frac{7}{16}$	6x4x$\frac{9}{16}$	61.12	2.66	5506	5529	9.49	9.51
*1847	" $\frac{5}{8}$	"	"	"	64.12	2.54	5670	5767	9.40	9.49
1848	" $\frac{11}{16}$	"	"	"	67.12	2.43	5832	6001	9.32	9.46
1849	" $\frac{3}{4}$	"	"	"	70.12	2.32	5994	6232	9.25	9.43
*1850	24x$\frac{9}{16}$	27x$\frac{5}{8}$	4x4x$\frac{7}{16}$	6x4x$\frac{5}{8}$	62.22	2.43	5702	5684	9.57	9.56
*1851	" $\frac{5}{8}$	"	"	"	65.22	2.32	5863	5922	9.48	9.53
1852	" $\frac{11}{16}$	"	"	"	68.22	2.22	6022	6156	9.40	9.50
1853	" $\frac{3}{4}$	"	"	"	71.22	2.12	6181	6387	9.32	9.47

* Spacing of rivet lines of web greater than 30 X thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

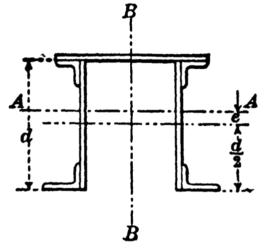
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1854	24x$\frac{9}{16}$	27x$\frac{5}{8}$	4x4x$\frac{7}{16}$	6x4x$\frac{11}{16}$	63.30	2.21	5883	5840	9.64	9.61
*1855	" $\frac{5}{8}$	"	"	"	66.30	2.11	6040	6078	9.55	9.58
1856	" $\frac{11}{16}$	"	"	"	69.30	2.02	6197	6312	9.46	9.55
1857	" $\frac{3}{4}$	"	"	"	72.30	1.93	6353	6543	9.38	9.51
*1858	24x$\frac{9}{16}$	27x$\frac{5}{8}$	4x4x$\frac{7}{16}$	6x4x$\frac{3}{4}$	64.38	1.99	6061	5994	9.71	9.66
*1859	" $\frac{5}{8}$	"	"	"	67.38	1.90	6217	6232	9.61	9.62
1860	" $\frac{11}{16}$	"	"	"	70.38	1.82	6371	6466	9.52	9.59
1861	" $\frac{3}{4}$	"	"	"	73.38	1.75	6524	6697	9.43	9.56
				24″ × 28″ Section. A Series.						
*1862	24x$\frac{5}{8}$	28x$\frac{5}{8}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{7}{16}$	5x3$\frac{1}{2}$x$\frac{1}{2}$	61.24	3.10	5190	6232	9.21	10.09
1863	" $\frac{11}{16}$	"	"	"	64.24	2.96	5361	6521	9.14	10.07
1864	" $\frac{3}{4}$	"	"	"	67.24	2.82	5531	6808	9.07	10.06
*1865	24x$\frac{5}{8}$	28x$\frac{5}{8}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{7}{16}$	5x3$\frac{1}{2}$x$\frac{9}{16}$	62.18	2.89	5372	6377	9.29	10.13
1866	" $\frac{11}{16}$	"	"	"	65.18	2.76	5539	6666	9.22	10.11
1867	" $\frac{3}{4}$	"	"	"	68.18	2.63	5707	6953	9.15	10.10
*1868	24x$\frac{5}{8}$	28x$\frac{5}{8}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{7}{16}$	5x3$\frac{1}{2}$x$\frac{5}{8}$	63.08	2.70	5540	6518	9.37	10.17
1869	" $\frac{11}{16}$	"	"	"	66.08	2.57	5706	6807	9.29	10.15
1870	" $\frac{3}{4}$	"	"	"	69.08	2.46	5869	7094	9.22	10.13
*1871	24x$\frac{5}{8}$	28x$\frac{5}{8}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{7}{16}$	5x3$\frac{1}{2}$x$\frac{11}{16}$	63.98	2.50	5705	6659	9.44	10.20
1872	" $\frac{11}{16}$	"	"	"	66.98	2.39	5866	6948	9.36	10.18
1873	" $\frac{3}{4}$	"	"	"	69.98	2.29	6027	7235	9.28	10.17
*1874	24x$\frac{5}{8}$	28x$\frac{5}{8}$	3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{7}{16}$	5x3$\frac{1}{2}$x$\frac{3}{4}$	64.86	2.32	5855	6791	9.50	10.23
1875	" $\frac{11}{16}$	"	"	"	67.86	2.22	6014	7080	9.42	10.21
1876	" $\frac{3}{4}$	"	"	"	70.86	2.13	6172	7367	9.34	10.19
				24″ × 28″ Section. B Series.						
*1877	24x$\frac{9}{16}$	28x$\frac{5}{8}$	4x4x$\frac{7}{16}$	6x4x$\frac{1}{2}$	60.62	3.01	5352	5930	9.39	9.89
*1878	" $\frac{5}{8}$	"	"	"	63.62	2.87	5522	6195	9.31	9.87
1879	" $\frac{11}{16}$	"	"	"	66.62	2.74	5690	6457	9.24	9.84
1880	" $\frac{3}{4}$	"	"	"	69.62	2.62	5855	6715	9.17	9.82

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 84.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Four Angles and Three Plates.

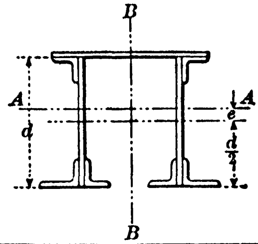
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*1881	24x9/16	28x5/8	4x4x7/16	6x4x9/16	61.74	2.76	5563	6100	9.49	9.94
*1882	" 5/8	"	"	"	64.74	2.63	5729	6365	9.41	9.92
1883	" 11/16	"	"	"	67.74	2.52	5892	6627	9.33	9.89
1884	" 3/4	"	"	"	70.74	2.41	6055	6885	9.25	9.86
*1885	24x9/16	28x5/8	4x4x7/16	6x4x5/8	62.84	2.53	5762	6268	9.58	9.99
*1886	" 5/8	"	"	"	65.84	2.41	5925	6533	9.49	9.96
1887	" 11/16	"	"	"	68.84	2.30	6086	6795	9.40	9.93
1888	" 3/4	"	"	"	71.84	2.21	6244	7053	9.32	9.91
*1889	24x9/16	28x5/8	4x4x7/16	6x4x11/16	63.92	2.30	5947	6437	9.65	10.03
*1890	" 5/8	"	"	"	66.92	2.20	6106	6702	9.55	10.00
1891	" 11/16	"	"	"	69.92	2.11	6263	6964	9.47	9.98
1892	" 3/4	"	"	"	72.92	2.02	6420	7222	9.39	9.95
*1893	24x9/16	28x5/8	4x4x7/16	6x4x3/4	65.00	2.09	6126	6604	9.71	10.08
*1894	" 5/8	"	"	"	68.00	2.00	6283	6869	9.61	10.05
1895	" 11/16	"	"	"	71.00	1.91	6439	7131	9.52	10.03
1896	" 3/4	"	"	"	74.00	1.83	6594	7389	9.44	10.00
				24″ × 30″ Section.						
*1897	24x5/8	30x11/16	4x4x3/8	6x4x1/2	65.85	3.22	5747	7465	9.35	10.65
1898	" 11/16	"	"	"	68.85	3.08	5921	7785	9.28	10.63
1899	" 3/4	"	"	"	71.85	2.95	6093	8103	9.21	10.62
*1900	24x5/8	30x11/16	4x4x3/8	6x4x9/16	66.97	2.99	5966	7663	9.44	10.70
1901	" 11/16	"	"	"	69.97	2.86	6136	7983	9.36	10.68
1902	" 3/4	"	"	"	72.97	2.74	6304	8301	9.29	10.66
*1903	24x5/8	30x11/16	4x4x3/8	6x4x5/8	68.07	2.76	6173	7859	9.52	10.74
1904	" 11/16	"	"	"	71.07	2.65	6339	8179	9.44	10.72
1905	" 3/4	"	"	"	74.07	2.54	6504	8497	9.37	10.71
*1906	24x5/8	30x11/16	4x4x3/8	6x4x11/16	69.15	2.56	6363	8056	9.59	10.79
1907	" 11/16	"	"	"	72.15	2.45	6526	8376	9.51	10.77
1908	" 3/4	"	"	"	75.15	2.35	6687	8694	9.43	10.75
*1909	24x5/8	30x11/16	4x4x3/8	6x4x3/4	70.23	2.35	6552	8250	9.67	10.84
1910	" 11/16	"	"	"	73.23	2.25	6712	8570	9.58	10.82
1911	" 3/4	"	"	"	76.23	2.17	6871	8888	9.49	10.80

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

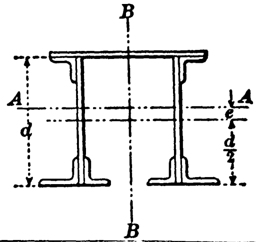
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom. Outside.	Angles. Bottom. Inside.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A–A. I_A	Moments of Inertia. Axis B–B. I_B	Radii of Gyration. Axis A–A. r_A	Radii of Gyration. Axis B–B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
					16″ × 20″ Section. A Series.						
*2001	$16x\frac{3}{8}$	$20x\frac{7}{16}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{3}{8}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{3}{8}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{3}{8}$	35.63	1.04	1553	1480	6.60	6.44
2002	“ $\frac{7}{16}$	“	“	“	“	37.63	0.98	1597	1551	6.51	6.41
2003	“ $\frac{1}{2}$	“	“	“	“	39.63	0.93	1642	1621	6.44	6.38
2004	“ $\frac{9}{16}$	“	“	“	“	41.63	0.89	1686	1689	6.36	6.36
2005	“ $\frac{5}{8}$	“	“	“	“	43.63	0.85	1730	1756	6.30	6.34
2006	“ $\frac{11}{16}$	“	“	“	“	45.63	0.81	1774	1821	6.24	6.31
2007	“ $\frac{3}{4}$	“	“	“	“	47.63	0.78	1818	1887	6.18	6.29
*2008	$16x\frac{3}{8}$	$20x\frac{7}{16}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{3}{8}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{7}{16}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{7}{16}$	37.19	0.72	1633	1547	6.63	6.44
2009	“ $\frac{7}{16}$	“	“	“	“	39.19	0.69	1677	1617	6.54	6.42
2010	“ $\frac{1}{2}$	“	“	“	“	41.19	0.66	1720	1686	6.46	6.40
2011	“ $\frac{9}{16}$	“	“	“	“	43.19	0.63	1763	1754	6.39	6.37
2012	“ $\frac{5}{8}$	“	“	“	“	45.19	0.60	1807	1821	6.32	6.34
2013	“ $\frac{11}{16}$	“	“	“	“	47.19	0.57	1850	1886	6.26	6.32
2014	“ $\frac{3}{4}$	“	“	“	“	49.19	0.55	1894	1951	6.20	6.30
*2015	$16x\frac{3}{8}$	$20x\frac{7}{16}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{3}{8}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{1}{2}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{1}{2}$	38.71	0.42	1729	1612	6.68	6.44
2016	“ $\frac{7}{16}$	“	“	“	“	40.71	0.41	1772	1682	6.60	6.42
2017	“ $\frac{1}{2}$	“	“	“	“	42.71	0.39	1815	1751	6.52	6.39
2018	“ $\frac{9}{16}$	“	“	“	“	44.71	0.38	1858	1819	6.44	6.37
2019	“ $\frac{5}{8}$	“	“	“	“	46.71	0.36	1901	1885	6.38	6.34
2020	“ $\frac{11}{16}$	“	“	“	“	48.71	0.34	1944	1949	6.32	6.32
2021	“ $\frac{3}{4}$	“	“	“	“	50.71	0.33	1987	2014	6.26	6.29
*2022	$16x\frac{3}{8}$	$20x\frac{7}{16}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{3}{8}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{9}{16}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{9}{16}$	40.19	0.16	1803	1675	6.70	6.45
2023	“ $\frac{7}{16}$	“	“	“	“	42.19	0.16	1845	1745	6.61	6.42
2024	“ $\frac{1}{2}$	“	“	“	“	44.19	0.15	1888	1813	6.53	6.39
2025	“ $\frac{9}{16}$	“	“	“	“	46.19	0.14	1931	1880	6.46	6.37
2026	“ $\frac{5}{8}$	“	“	“	“	48.19	0.13	1973	1946	6.40	6.35
2027	“ $\frac{11}{16}$	“	“	“	“	50.19	0.12	2016	2010	6.34	6.32
2028	“ $\frac{3}{4}$	“	“	“	“	52.19	0.12	2059	2074	6.28	6.29
*2029	$16x\frac{3}{8}$	$20x\frac{7}{16}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{3}{8}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{5}{8}$	$3\frac{1}{2}x3\frac{1}{2}x\frac{5}{8}$	41.63	−.08	1870	1738	6.70	6.46
2030	“ $\frac{7}{16}$	“	“	“	“	43.63	−.08	1913	1807	6.62	6.44
2031	“ $\frac{1}{2}$	“	“	“	“	45.63	−.07	1956	1874	6.54	6.41
2032	“ $\frac{9}{16}$	“	“	“	“	47.63	−.07	1998	1941	6.47	6.38
2033	“ $\frac{5}{8}$	“	“	“	“	49.63	−.07	2041	2007	6.41	6.36
2034	“ $\frac{11}{16}$	“	“	“	“	51.63	−.07	2084	2070	6.35	6.34
2035	“ $\frac{3}{4}$	“	“	“	“	53.63	−.06	2126	2134	6.30	6.32

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Six Angles and Three Plates.

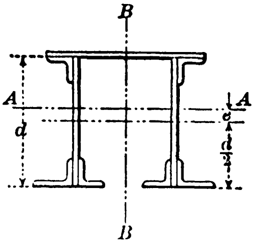
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom. Outside.	Angles. Bottom. Inside.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
					16″ × 20″ Section. B Series.						
*2036	16x3/8	20x7/16	3½x3½x3/8	5x3½x3/8	3½x3½x3/8	36.77	0.77	1640	1606	6.67	6.61
2037	" 7/16	"	"	"	"	38.77	0.73	1684	1677	6.59	6.58
2038	" ½	"	"	"	"	40.77	0.70	1727	1747	6.51	6.55
2039	" 9/16	"	"	"	"	42.77	0.67	1771	1815	6.43	6.52
2040	" 5/8	"	"	"	"	44.77	0.64	1814	1882	6.36	6.48
2041	" 11/16	"	"	"	"	46.77	0.61	1858	1947	6.30	6.45
2042	" ¾	"	"	"	"	48.77	0.58	1902	2013	6.24	6.42
*2043	16x3/8	20x7/16	3½x3½x3/8	5x3½x7/16	3½x3½x7/16	38.51	0.43	1725	1695	6.69	6.63
2044	" 7/16	"	"	"	"	40.51	0.42	1768	1765	6.60	6.60
2045	" ½	"	"	"	"	42.51	0.40	1810	1834	6.52	6.57
2046	" 9/16	"	"	"	"	44.51	0.38	1854	1902	6 45	6.54
2047	" 5/8	"	"	"	"	46.51	0.36	1897	1970	6.39	6.51
2048	" 11/16	"	"	"	"	48.51	0.34	1940	2034	6.32	6.48
2049	" ¾	"	"	"	"	50.51	0.33	1982	2099	6.26	6.45
*2050	16x3/8	20x7/16	3½x3½x3/8	5x3½x½	3½x3½x½	40.21	0.12	1826	1781	6.74	6.65
2051	" 7/16	"	"	"	"	42.21	0.12	1868	1852	6.65	6.62
2052	" ½	"	"	"	"	44.21	0.11	1911	1920	6.57	6.58
2053	" 9/16	"	"	"	"	46.21	0.11	1954	1988	6.50	6.55
2054	" 5/8	"	"	"	"	48.21	0.11	1996	2054	6.43	6.52
2055	" 11/16	"	"	"	"	50.21	0.10	2039	2119	6.37	6.49
2056	" ¾	"	"	"	"	52.21	0.10	2082	2183	6.31	6.46
*2057	16x3/8	20x7/16	3½x3½x3/8	5x3½x9/16	3½x3½x9/16	41.89	—.15	1903	1866	6.75	6.67
2058	" 7/16	"	"	"	"	43.89	—.14	1946	1936	6.66	6.64
2059	" ½	"	"	"	"	45.89	—.14	1988	2004	6.58	6.61
2060	" 9/16	"	"	"	"	47.89	—.13	2031	2071	6.51	6.58
2061	" 5/8	"	"	"	"	49.89	—.13	2074	2137	6.45	6.55
2062	" 11/16	"	"	"	"	51.89	—.12	2115	2201	6.39	6.52
2063	" ¾	"	"	"	"	53.89	—.12	2158	2265	6.32	6.48
*2064	16x3/8	20x7/16	3½x3½x3/8	5x3½x5/8	3½x3½x5/8	43.51	—.41	1978	1951	6.74	6.70
2065	" 7/16	"	"	"	"	45.51	—.39	2021	2020	6.65	6.66
2066	" ½	"	"	"	"	47.51	—.37	2063	2087	6.58	6.63
2067	" 9/16	"	"	"	"	49.51	—.36	2107	2154	6.52	6.60
2068	" 5/8	"	"	"	"	51.51	—.34	2150	2220	6.46	6.57
2069	" 11/16	"	"	"	"	53.51	—.33	2192	2283	6.40	6.53
2070	" ¾	"	"	"	"	55.51	—.32	2235	2347	6.34	6.50

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

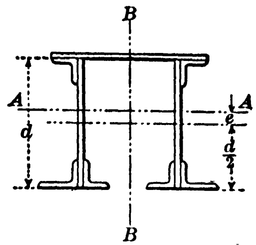
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom. Outside.	Angles. Bottom. Inside.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
					16″ × 22″ Section.						
*2071	16x3/8	22x1/2	3 1/2x3 1/2x3/8	5x3 1/2x3/8	3 1/2x3 1/2x3/8	39.02	1.21	1761	2163	6.72	7.45
2072	“ 7/16	“	“	“	“	41.02	1.15	1807	2259	6.64	7.42
2073	“ 1/2	“	“	“	“	43.02	1.10	1851	2354	6.56	7.40
2074	“ 9/16	“	“	“	“	45.02	1.05	1897	2448	6.49	7.37
2075	“ 5/8	“	“	“	“	47.02	1.00	1942	2540	6.43	7.35
2076	“ 11/16	“	“	“	“	49.02	0.96	1988	2630	6.37	7.33
2077	“ 3/4	“	“	“	“	51.02	0.92	2031	2718	6.31	7.30
*2078	16x3/8	22x1/2	3 1/2x3 1/2x3/8	5x3 1/2x7/16	3 1/2x3 1/2x7/16	40.76	0.86	1873	2276	6.78	7.47
2079	“ 7/16	“	“	“	“	42.76	0.82	1917	2372	6.70	7.45
2080	“ 1/2	“	“	“	“	44.76	0.78	1960	2467	6.62	7.43
2081	“ 9/16	“	“	“	“	46.76	0.75	2005	2560	6.55	7.40
2082	“ 5/8	“	“	“	“	48.76	0.72	2049	2652	6.48	7.38
2083	“ 11/16	“	“	“	“	50.76	0.69	2093	2741	6.42	7.35
2084	“ 3/4	“	“	“	“	52.76	0.67	2136	2828	6.36	7.32
*2085	16x3/8	22x1/2	3 1/2x3 1/2x3/8	5x3 1/2x1/2	3 1/2x3 1/2x1/2	42.46	0.56	1970	2388	6.81	7.50
2086	“ 7/16	“	“	“	“	44.46	0.53	2013	2483	6.73	7.47
2087	“ 1/2	“	“	“	“	46.46	0.51	2056	2577	6.65	7.45
2088	“ 9/16	“	“	“	“	48.46	0.49	2099	2670	6.59	7.42
2089	“ 5/8	“	“	“	“	50.46	0.47	2142	2761	6.52	7.40
2090	“ 11/16	“	“	“	“	52.46	0.45	2186	2850	6.45	7.37
2091	“ 3/4	“	“	“	“	54.46	0.43	2229	2937	6.40	7.35
*2092	16x3/8	22x1/2	3 1/2x3 1/2x3/8	5x3 1/2x9/16	3 1/2x3 1/2x9/16	44.14	0.27	2060	2498	6.83	7.52
2093	“ 7/16	“	“	“	“	46.14	0.26	2103	2593	6.75	7.50
2094	“ 1/2	“	“	“	“	48.14	0.25	2145	2687	6.68	7.47
2095	“ 9/16	“	“	“	“	50.14	0.24	2188	2779	6.61	7.44
2096	“ 5/8	“	“	“	“	52.14	0.23	2231	2869	6.54	7.42
2097	“ 11/16	“	“	“	“	54.14	0.22	2274	2957	6.48	7.39
2098	“ 3/4	“	“	“	“	56.14	0.22	2316	3043	6.42	7.36
*2099	16x3/8	22x1/2	3 1/2x3 1/2x3/8	5x3 1/2x5/8	3 1/2x3 1/2x5/8	45.76	0.02	2139	2605	6.84	7.55
2100	“ 7/16	“	“	“	“	47.76	0.02	2182	2699	6.76	7.52
2101	“ 1/2	“	“	“	“	49.76	0.02	2224	2792	6.69	7.49
2102	“ 9/16	“	“	“	“	51.76	0.02	2267	2883	6.62	7.46
2103	“ 5/8	“	“	“	“	53.76	0.02	2310	2973	6.56	7.44
2104	“ 11/16	“	“	“	“	55.76	0.02	2353	3061	6.50	7.41
2105	“ 3/4	“	“	“	“	57.76	0.02	2395	3147	6.44	7.38

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

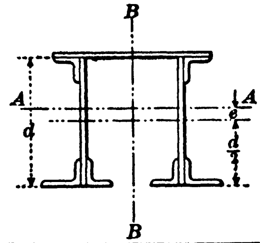
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom. Outside.	Angles. Bottom. Inside.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A–A. I_A	Moments of Inertia. Axis B–B. I_B	Radii of Gyration. Axis A–A. r_A	Radii of Gyration. Axis B–B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2106	16x3/8	22x1/2	3½x3½x3/8	5x3½x11/16	3½x3½x11/16	47.38	−.21	2212	2712	6.83	7.56
2107	“ 7/16	“	“	“	“	49.38	−.20	2255	2806	6.76	7.54
2108	“ 1/2	“	“	“	“	51.38	−.19	2297	2899	6.69	7.51
2109	“ 9/16	“	“	“	“	53.38	−.18	2340	2989	6.62	7.48
2110	“ 5/8	“	“	“	“	55.38	−.18	2383	3078	6.56	7.45
2111	“ 11/16	“	“	“	“	57.38	−.17	2426	3165	6.50	7.43
2112	“ 3/4	“	“	“	“	59.38	−.16	2468	3251	6.45	7.40
*2113	16x3/8	22x1/2	3½x3½x3/8	5x3½x3/4	3½x3½x3/4	48.96	−.41	2275	2817	6.83	7.59
2114	“ 7/16	“	“	“	“	50.96	−.40	2318	2910	6.74	7.56
2115	“ 1/2	“	“	“	“	52.96	−.38	2360	3002	6.67	7.53
2116	“ 9/16	“	“	“	“	54.96	−.37	2404	3092	6.61	7.50
2117	“ 5/8	“	“	“	“	56.96	−.35	2447	3181	6.55	7.47
2118	“ 11/16	“	“	“	“	58.96	−.34	2492	3268	6.50	7.44
2119	“ 3/4	“	“	“	“	60.96	−.33	2532	3353	6.44	7.41
18″ × 22″ Section. A Series.											
*2120	18x3/8	22x1/2	3½x3½x3/8	3½x3½x3/8	3½x3½x3/8	39.38	1.58	2177	2086	7.43	7.28
*2121	“ 7/16	“	“	“	“	41.63	1.49	2243	2196	7.34	7.26
2122	“ 1/2	“	“	“	“	43.88	1.41	2309	2304	7.25	7.24
2123	“ 9/16	“	“	“	“	46.13	1.34	2374	2410	7.17	7.23
2124	“ 5/8	“	“	“	“	48.38	1.28	2439	2514	7.10	7.21
2125	“ 11/16	“	“	“	“	50.63	1.23	2503	2616	7.03	7.19
2126	“ 3/4	“	“	“	“	52.88	1.17	2566	2716	6.96	7.16
*2127	18x3/8	22x1/2	3½x3½x3/8	3½x3½x7/16	3½x3½x7/16	40.94	1.22	2310	2176	7.51	7.29
*2128	“ 7/16	“	“	“	“	43.19	1.16	2374	2285	7.41	7.28
2129	“ 1/2	“	“	“	“	45.44	1.10	2437	2393	7.32	7.26
2130	“ 9/16	“	“	“	“	47.69	1.05	2500	2500	7.24	7.24
2131	“ 5/8	“	“	“	“	49.94	1.00	2564	2604	7.17	7.22
2132	“ 11/16	“	“	“	“	52.19	0.96	2627	2703	7.10	7.20
2133	“ 3/4	“	“	“	“	54.44	0.92	2689	2802	7.03	7.18
*2134	18x3/8	22x1/2	3½x3½x3/8	3½x3½x1/2	3½x3½x1/2	42.46	0.90	2428	2259	7.56	7.29
*2135	“ 7/16	“	“	“	“	44.71	0.85	2491	2368	7.46	7.28
2136	“ 1/2	“	“	“	“	46.96	0.81	2553	2475	7.37	7.26
2137	“ 9/16	“	“	“	“	49.21	0.77	2616	2581	7.29	7.24
2138	“ 5/8	“	“	“	“	51.46	0.74	2678	2683	7.21	7.22
2139	“ 11/16	“	“	“	“	53.71	0.71	2740	2784	7.14	7.20
2140	“ 3/4	“	“	“	“	55.96	0.68	2801	2883	7.08	7.18

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

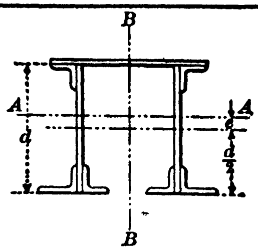
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom. Outside.	Angles. Bottom. Inside.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2141	18x⅜	22x½	3½x3½x⅜	3½x3½x9/16	3½x3½x9/16	43.94	0.60	2538	2345	7.60	7.30
*2142	" 7/16	"	"	"	"	46.19	0.57	2600	2454	7.51	7.29
2143	" ½	"	"	"	"	48.44	0.55	2660	2559	7.42	7.27
2144	" 9/16	"	"	"	"	50.69	0.52	2722	2665	7.34	7.25
2145	" ⅝	"	"	"	"	52.94	0.50	2785	2765	7.26	7.23
2146	" 11/16	"	"	"	"	55.19	0.48	2845	2866	7.18	7.21
2147	" ¾	"	"	"	"	57.44	0.46	2906	2966	7.11	7.19
*2148	18x⅜	22x½	3½x3½x⅜	3½x3½x⅝	3½x3½x⅝	45.38	0.34	2636	2426	7.62	7.31
*2149	" 7/16	"	"	"	"	47.63	0.32	2697	2535	7.53	7.29
2150	" ½	"	"	"	"	49.88	0.31	2757	2640	7.44	7.27
2151	" 9/16	"	"	"	"	52.13	0.30	2818	2744	7.35	7.25
2152	" ⅝	"	"	"	"	54.38	0.29	2879	2846	7 27	7.23
2153	" 11/16	"	"	"	"	56.63	0.37	2940	2947	7.20	7.21
2154	" ¾	"	"	"	"	58.88	0.36	3001	3044	7.14	7.19
*2155	18x⅜	22x½	3½x3½x⅜	3½x3½x11/16	3½x3½x11/16	46.82	0.12	2722	2506	7.63	7.32
*2156	" 7/16	"	"	"	"	49.07	0.11	2783	2613	7.53	7.30
2157	" ½	"	"	"	"	51.32	0.11	2843	2719	7.44	7.28
2158	" 9/16	"	"	"	"	53.57	0.10	2904	2824	7.36	7.26
2159	" ⅝	"	"	"	"	55.82	0.10	2965	2924	7.29	7.24
2160	" 11/16	"	"	"	"	58.07	0.09	3025	3024	7.22	7.22
2161	" ¾	"	"	"	"	60.32	0.09	3086	3122	7.15	7.20
*2162	18x⅜	22x½	3½x3½x⅜	3½x3¼x¾	3½x3½x¾	48.22	−.11	2802	2585	7.62	7.32
*2163	" 7/16	"	"	"	"	50.47	−.11	2863	2693	7.53	7.30
2164	" ½	"	"	"	"	52.72	−.10	2923	2797	7.44	7.28
2165	" 9/16	"	"	"	"	54.97	−.10	2984	2902	7.36	7.26
2166	" ⅝	"	"	"	"	57.22	−.10	3045	3001	7.29	7.24
2167	" 11/16	"	"	"	"	59.47	−.09	3105	3101	7.22	7.22
2168	" ¾	"	"	"	"	61.72	−.09	3166	3198	7.16	7.20
18″ × 22″ Section. B Series.											
*2169	18x⅜	22x½	3½x3½x⅜	5x3½x⅜	3½x3½x⅜	40.52	1.29	2297	2241	7.53	7.44
*2170	" 7/16	"	"	"	"	42.77	1.22	2361	2351	7.43	7.42
2171	" ½	"	"	"	"	45.02	1.16	2426	2459	7.34	7.39
2172	" 9/16	"	"	"	"	47.27	1.10	2489	2566	7.26	7.37
2173	" ⅝	"	"	"	"	49.52	1.05	2552	2669	7.18	7.34
2174	" 11/16	"	"	"	"	51.77	1.01	2615	2772	7.11	7.32
2175	" ¾	"	"	"	"	54.02	0.97	2678	2872	7.04	7.29

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



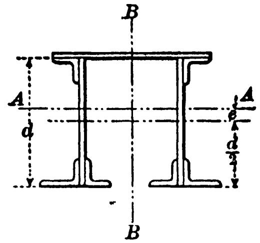
Six Angles and Three Plates.

Section Number.	Plates.		Angles.			Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom. Outside.	Bottom. Inside.	A	e	Axis A-A. I_A	Axis B-B. I_B	Axis A-A. r_A	Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2176	18x3/8	22x1/2	3½x3½x3/8	5x3½x7/16	3½x3½x7/16	42.26	0.90	2437	2357	7.60	7.47
*2177	" 7/16	"	"	"	"	44.51	0.86	2500	2467	7.50	7.44
2178	" 1/2	"	"	"	"	46.76	0.82	2563	2574	7.41	7.42
2179	" 9/16	"	"	"	"	49.01	0.78	2624	2681	7.33	7.39
2180	" 5/8	"	"	"	"	51.26	0.75	2684	2783	7.25	7.37
2181	" 11/16	"	"	"	"	53.51	0.72	2746	2885	7.17	7.34
2182	" 3/4	"	"	"	"	55.76	0.69	2810	2985	7.10	7.31
*2183	18x3/8	22x1/2	3½x3½x3/8	5x3½x½	3½x3½x½	43.96	0.57	2563	2466	7.64	7.49
*2184	" 7/16	"	"	"	"	46.21	0.55	2623	2575	7.54	7.47
2185	" 1/2	"	"	"	"	48.46	0.52	2685	2682	7.45	7.44
2186	" 9/16	"	"	"	"	50.71	0.50	2745	2788	7.36	7.41
2187	" 5/8	"	"	"	"	52.96	0.48	2807	2890	7.28	7.39
2188	" 11/16	"	"	"	"	55.21	0.46	2868	2991	7.21	7.36
2189	" 3/4	"	"	"	"	57.46	0.44	2930	3090	7.14	7.34
*2190	18x3/8	22x1/2	3½x3½x3/8	5x3½x9/16	3½x3½x9/16	45.64	0.26	2680	2578	7.66	7.52
*2191	" 7/16	"	"	"	"	47.89	0.25	2741	2687	7.56	7.49
2192	" 1/2	"	"	"	"	50.14	0.24	2801	2792	7.47	7.46
2193	" 9/16	"	"	"	"	52.39	0.23	2862	2898	7.39	7.44
2194	" 5/8	"	"	"	"	54.64	0.22	2923	2998	7.31	7.41
2195	" 11/16	"	"	"	"	56.89	0.21	2984	3101	7.24	7.38
2196	" 3/4	"	"	"	"	59.14	0.20	3045	3199	7.18	7.36
*2197	18x3/8	22x1/2	3½x3½x3/8	5x3½x5/8	3½x3½x5/8	47.26	−.02	2782	2685	7.67	7.54
*2198	" 7/16	"	"	"	"	49.51	−.02	2843	2794	7.57	7.51
2199	" 1/2	"	"	"	"	51.76	−.02	2904	2899	7.48	7.48
2200	" 9/16	"	"	"	"	54.01	−.01	2964	3003	7.40	7.46
2201	" 5/8	"	"	"	"	56.26	−.01	3025	3105	7.33	7.43
2202	" 11/16	"	"	"	"	58.51	−.01	3086	3206	7.26	7.40
2203	" 3/4	"	"	"	"	60.76	−.01	3146	3303	7.20	7.37
*2204	18x3/8	22x1/2	3½x3½x3/8	5x3½x11/16	3½x3½x11/16	48.88	−.27	2875	2791	7.67	7.56
*2205	" 7/16	"	"	"	"	51.13	−.26	2937	2898	7.57	7.53
2206	" 1/2	"	"	"	"	53.38	−.25	2998	3004	7.48	7.50
2207	" 9/16	"	"	"	"	55.63	−.24	3059	3109	7.41	7.48
2208	" 5/8	"	"	"	"	57.88	−.23	3119	3209	7.34	7.45
2209	" 11/16	"	"	"	"	60.13	−.22	3180	3309	7.27	7.42
2210	" 3/4	"	"	"	"	62.38	−.21	3241	3407	7.20	7.39

* Spacing of rivet lines of web greater than 30 × thickness of plate.

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

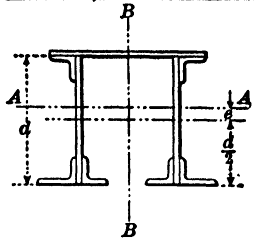
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom. Outside.	Angles. Bottom. Inside.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2211	18x⅜	22x½	3½x3½x⅜	5x3½x¾	3½x3½x¾	50.46	−.50	2958	2896	7.65	7.57
*2212	“ 7/16	“	“	“	“	52.71	−.48	3020	3003	7.55	7.54
2213	“ ½	“	“	“	“	54.96	−.46	3081	3108	7.47	7.52
2214	“ 9/16	“	“	“	“	57.21	−.44	3142	3212	7.40	7.49
2215	“ ⅝	“	“	“	“	59.46	−.42	3203	3312	7.33	7.47
2216	“ 11/16	“	“	“	“	61.71	−.41	3265	3412	7.26	7.44
2217	“ ¾	“	“	“	“	63.96	−.39	3326	3508	7.20	7.41
18″ × 24″ Section.											
2218	18x½	24x9/16	3½x3½x⅜	5x3½x⅜	3½x3½x⅜	47.52	1.59	2584	3215	7.37	8.23
2219	“ 9/16	“	“	“	“	49.77	1.52	2650	3354	7.29	8.21
2220	“ ⅝	“	“	“	“	52.02	1.45	2716	3491	7.22	8.19
2221	“ 11/16	“	“	“	“	54.27	1.39	2781	3625	7.16	8.17
2222	“ ¾	“	“	“	“	56.52	1.34	2846	3757	7.10	8.15
2223	18x½	24x9/16	3½x3½x⅜	5x3½x7/16	3½x3½x7/16	49.26	1.26	2736	3354	7.45	8.25
2224	“ 9/16	“	“	“	“	51.51	1.20	2801	3492	7.37	8.23
2225	“ ⅝	“	“	“	“	53.76	1.15	2865	3628	7.30	8.21
2226	“ 11/16	“	“	“	“	56.01	1.10	2928	3761	7.23	8.19
2227	“ ¾	“	“	“	“	58.26	1.06	2991	3893	7.17	8.17
2228	18x½	24x9/16	3½x3½x⅜	5x3½x½	3½x3½x½	50.96	0.95	2874	3494	7.51	8.28
2229	“ 9/16	“	“	“	“	53.21	0.91	2937	3632	7.43	8.26
2230	“ ⅝	“	“	“	“	55.46	0.88	2999	3767	7.36	8.24
2231	“ 11/16	“	“	“	“	57.71	0.84	3061	3900	7.28	8.22
2232	“ ¾	“	“	“	“	59.96	0.81	3124	4031	7.22	8.20
2233	18x½	24x9/16	3½x3½x⅜	5x3½x9/16	3½x3½x9/16	52.64	0.67	3001	3631	7.55	8.31
2234	“ 9/16	“	“	“	“	54.89	0.64	3063	3768	7.47	8.28
2235	“ ⅝	“	“	“	“	57.14	0.62	3125	3903	7.39	8.26
2236	“ 11/16	“	“	“	“	59.39	0.60	3186	4035	7.32	8.24
2237	“ ¾	“	“	“	“	61.64	0.57	3248	4165	7.26	8.22
2238	18x½	24x9/16	3½x3½x⅜	5x3½x⅝	3½x3½x⅝	54.26	0.42	3114	3766	7.58	8.33
2239	“ 9/16	“	“	“	“	56.51	0.40	3176	3902	7.50	8.31
2240	“ ⅝	“	“	“	“	58.76	0.39	3237	4036	7.42	8.29
2241	“ 11/16	“	“	“	“	61.01	0.37	3297	4168	7.35	8.26
2242	“ ¾	“	“	“	“	63.26	0.36	3359	4298	7.29	8.24

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Six Angles and Three Plates.

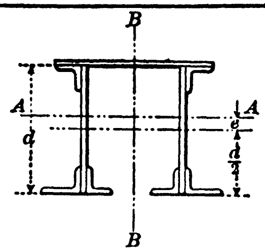
Section Number.	Plates.		Angles.			Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.				Axis A–A.	Axis B–B.	Axis A–A.	Axis B–B.
				Outside.	Inside.	A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
2243	18x½	24x9/16	3½x3½x⅜	5x3½x11/16	3½x3½x11/16	55.88	0.18	3221	3895	7.59	8.35
2244	" 9/16	"	"	"	"	58.13	0.18	3282	4031	7.51	8.33
2245	" ⅝	"	"	"	"	60.38	0.17	3343	4165	7.44	8.31
2246	" 11/16	"	"	"	"	62.63	0.16	3403	4296	7.37	8.28
2247	" ¾	"	"	"	"	64.88	0.16	3464	4425	7.31	8.26
2248	18x½	24x9/16	3½x3½x⅜	5x3½x¾	3½x3½x¾	57.46	—.03	3314	4026	7.60	8.37
2249	" 9/16	"	"	"	"	59.71	—.03	3375	4161	7.52	8.35
2250	" ⅝	"	"	"	"	61.96	—.03	3436	4294	7.45	8.33
2251	" 11/16	"	"	"	"	64.21	—.03	3496	4424	7.38	8.30
2252	" ¾	"	"	"	"	66.46	—.03	3557	4553	7.32	8.28
20″ × 24″ Section. A Series.											
*2253	20x½	24x9/16	3½x3½x⅜	3½x3½x⅜	3½x3½x⅜	48.38	1.94	3136	3171	8.04	8.09
2254	" 9/16	"	"	"	"	50.88	1.85	3227	3324	7.96	8.08
2255	" ⅝	"	"	"	"	53.38	1.76	3319	3477	7.88	8.06
2256	" 11/16	"	"	"	"	55.88	1.68	3410	3627	7.81	8.05
2257	" ¾	"	"	"	"	58.38	1.61	3500	3777	7.74	8.04
*2258	20x½	24x9/16	3½x3½x⅜	3½x3½x7/16	3½x3½x7/16	49.94	1.61	3310	3282	8.14	8.10
2259	" 9/16	"	"	"	"	52.44	1.53	3400	3435	8.05	8.09
2260	" ⅝	"	"	"	"	54.94	1.46	3489	3587	7.96	8.08
2261	" 11/16	"	"	"	"	57.44	1.40	3577	3736	7.88	8.06
2262	" ¾	"	"	"	"	59.94	1.34	3665	3886	7.82	8.05
*2263	20x½	24x9/16	3½x3½x⅜	3½x3½x½	3½x3½x½	51.46	1.31	3466	3387	8.21	8.12
2264	" 9/16	"	"	"	"	53.96	1.25	3553	3540	8.12	8.10
2265	" ⅝	"	"	"	"	56.46	1.19	3640	3691	8.03	8.09
2266	" 11/16	"	"	"	"	58.96	1.14	3728	3839	7.95	8.07
2267	" ¾	"	"	"	"	61.46	1.09	3815	3988	7.89	8.05
*2268	20x½	24x9/16	3½x3½x⅜	3½x3½x9/16	3½x3½x9/16	52.94	1.02	3617	3497	8.26	8.13
2269	" 9/16	"	"	"	"	55.44	0.97	3703	3649	8.17	8.11
2270	" ⅝	"	"	"	"	57.94	0.93	3788	3799	8.08	8.09
2271	" 11/16	"	"	"	"	60.44	0.89	3874	3947	8.00	8.08
2272	" ¾	"	"	"	"	62.94	0.86	3959	4095	7.93	8.06

*Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

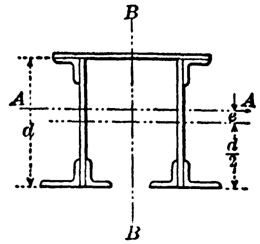
Section Number.	Plates.		Angles.			Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.				Axis A-A.	Axis B-B.	Axis A-A.	Axis B-B.
				Outside.	Inside.	A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2273	20x½	24x9/16	3½x3½x3/8	3½x3½x5/8	3½x3½x5/8	54.38	0.76	3752	3599	8.30	8.13
2274	" 9/16	"	"	"	"	56.88	0.73	3836	3751	8.21	8.11
2275	" 5/8	"	"	"	"	59.38	0.70	3921	3900	8.12	8.10
2276	" 11/16	"	"	"	"	61.88	0.67	4005	4047	8.04	8.08
2277	" ¾	"	"	"	"	64.38	0.64	4090	4195	7.97	8.07
*2278	20x½	24x9/16	3½x3½x3/8	3½x3½x11/16	3½x3½x11/16	55.82	0.53	3873	3700	8.33	8.14
2279	" 9/16	"	"	"	"	58.32	0.50	3957	3851	8.23	8.12
2280	" 5/8	"	"	"	"	60.82	0.48	4041	4000	8.14	8.10
2281	" 11/16	"	"	"	"	63.32	0.46	4115	4147	8.06	8.08
2282	" ¾	"	"	"	"	65.82	0.45	4209	4294	7.99	8.07
*2283	20x½	24x9/16	3½x3½x3/8	3½x3½x¾	3½x3½x¾	57.22	0.30	3985	3800	8.35	8.15
2284	" 9/16	"	"	"	"	59.72	0.29	4068	3951	8.25	8.13
2285	" 5/8	"	"	"	"	62.22	0.28	4151	4099	8.16	8.11
2286	" 11/16	"	"	"	"	64.72	0.27	4235	4245	8.08	8 09
2287	" ¾	"	"	"	"	67.22	0.26	4319	4392	8.01	8.08
				20″ × 24″ Section. B Series.							
*2288	20x½	24x9/16	3½x3½x3/8	5x3½x3/8	3½x3½x3/8	49.52	1.67	3285	3354	8.14	8.22
2289	" 9/16	"	"	"	"	52.02	1.59	3375	3507	8.05	8.20
2290	" 5/8	"	"	"	"	54.52	1.52	3465	3660	7.97	8.19
2291	" 11/16	"	"	"	"	57.02	1.45	3554	3810	7.89	8.17
2292	" ¾	"	"	"	"	59.52	1.39	3642	3960	7.82	8.15
*2293	20x½	24x9/16	3½x3½x3/8	5x3½x7/16	3½x3½x7/16	51.26	1.33	3473	3495	8.23	8.25
2294	" 9/16	"	"	"	"	53.76	1.27	3560	3648	8.14	8.23
2295	" 5/8	"	"	"	"	56.26	1.21	3648	3800	8.05	8.22
2296	" 11/16	"	"	"	"	58.76	1.16	3734	3949	7.97	8.20
2297	" ¾	"	"	"	"	61.26	1.11	3820	4099	7.90	8.18
*2298	20x½	24x9/16	3½x3½x3/8	5x3½x½	3½x3½x½	52.96	0.98	3644	3631	8.30	8.28
2299	" 9/16	"	"	"	"	55.46	0.93	3732	3784	8.20	8.26
2300	" 5/8	"	"	"	"	57.96	0.90	3817	3935	8.11	8.23
2301	" 11/16	"	"	"	"	60.46	0.86	3902	4083	8.03	8.21
2302	" ¾	"	"	"	"	62.96	0.83	3988	4232	7.96	8.19

*Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Six Angles and Three Plates.

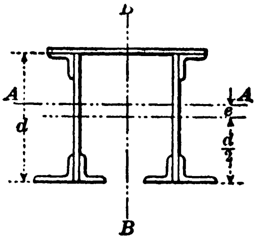
Section Number.	Plates.		Angles.			Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.				Axis A-A.	Axis B-B.	Axis A-A.	Axis B-B.
				Outside.	Inside.	A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2303	20x½	24x9/16	3½x3½x⅜	5x3½x9/16	3½x3½x9/16	54.64	0.69	3807	3771	8.34	8.30
2304	“ 9/16	“	“	“	“	57.14	0.66	3891	3923	8.25	8.28
2305	“ ⅝	“	“	“	“	59.64	0.63	3975	4073	8.16	8.26
2306	“ 11/16	“	“	“	“	62.14	0.61	4059	4221	8.07	8.24
2307	“ ¾	“	“	“	“	64.64	0.59	4143	4369	8.00	8.22
*2308	20x½	24x9/16	3½x3½x⅜	5x3½x⅝	3½x3½x⅝	56.26	0.42	3949	3904	8.38	8.33
2309	“ 9/16	“	“	“	“	58.76	0.40	4033	4056	8.29	8.31
2310	“ ⅝	“	“	“	“	61.26	0.38	4117	4205	8.20	8.28
2311	“ 11/16	“	“	“	“	63.76	0.36	4208	4352	8.12	8.26
2312	“ ¾	“	“	“	“	66.26	0.34	4284	4500	8.04	8.24
*2313	20x½	24x9/16	3½x3½x⅜	5x3½x11/16	3½x3½x11/16	57.88	0.16	4081	4036	8.40	8.35
2314	“ 9/16	“	“	“	“	60.38	0.15	4164	4186	8.30	8.33
2315	“ ⅝	“	“	“	“	62.88	0.15	4247	4336	8.22	8.31
2316	“ 11/16	“	“	“	“	65.38	0.14	4331	4483	8.14	8.28
2317	“ ¾	“	“	“	“	67.88	0.14	4414	4630	8.06	8.26
*2318	20x½	24x9/16	3½x3½x⅜	5x3½x¾	3½x3½x¾	59.46	−.07	4200	4166	8.40	8.37
2319	“ 9/16	“	“	“	“	61.96	−.07	4283	4317	8.31	8.35
2320	“ ⅝	“	“	“	“	64.46	−.06	4366	4465	8.23	8.32
2321	“ 11/16	“	“	“	“	66.96	−.06	4450	4611	8.15	8.30
2322	“ ¾	“	“	“	“	69.46	−.05	4533	4758	8.08	8.28
20″ × 26″ Section.											
*2323	20x½	26x⅝	3½x3½x⅜	5x3½x⅜	3½x3½x⅜	52.27	2.14	3485	4272	8.16	9.04
2324	“ 9/16	“	“	“	“	54.77	2.04	3579	4468	8.08	9.03
2325	“ ⅝	“	“	“	“	57.27	1.95	3673	4661	8.01	9.02
2326	“ 11/16	“	“	“	“	59.77	1.87	3765	4851	7.94	9.01
2327	“ ¾	“	“	“	“	62.27	1.79	3856	5039	7.87	8.99
*2328	20x½	26x⅝	3½x3½x⅜	5x3½x7/16	3½x3½x7/16	54.01	1.78	3694	4443	8.27	9.07
2329	“ 9/16	“	“	“	“	56.51	1.71	3783	4638	8.18	9.06
2330	“ ⅝	“	“	“	“	59.01	1.63	3874	4831	8.10	9.05
2331	“ 11/16	“	“	“	“	61.51	1.57	3963	5020	8.03	9.04
2332	“ ¾	“	“	“	“	64.01	1.51	4052	5207	7.96	9.02

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

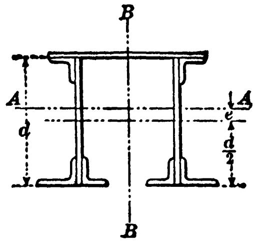
Section Number.	Plates.		Angles.			Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.				Axis A-A.	Axis B-B.	Axis A-A.	Axis B-B.
				Outside.	Inside.	A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2333	20x½	26x⅝	3½x3½x⅜	5x3½x½	3½x3½x½	55.71	1.46	3879	4614	8.35	9.10
2334	" 9/16	"	"	"	"	58.21	1.40	3967	4809	8.26	9.09
2335	" ⅝	"	"	"	"	60.71	1.34	4056	5000	8.17	9.08
2336	" 11/16	"	"	"	"	63.21	1.29	4143	5189	8.10	9.06
2337	" ¾	"	"	"	"	65.71	1.24	4230	5375	8.02	9.04
*2338	20x½	26x⅝	3½x3½x⅜	5x3½x9/16	3½x3½x9/16	57.39	1.16	4053	4782	8.40	9.13
2339	" 9/16	"	"	"	"	59.89	1.11	4139	4976	8.31	9.11
2340	" ⅝	"	"	"	"	62.39	1.06	4226	5167	8.23	9.10
2341	" 11/16	"	"	"	"	64.89	1.02	4312	5355	8.15	9.08
2342	" ¾	"	"	"	"	67.39	0.99	4397	5541	8.08	9.07
*2343	20x½	26x⅝	3½x3½x⅜	5x3½x⅝	3½x3½x⅝	59.01	0.89	4211	4945	8.45	9.15
2344	" 9/16	"	"	"	"	61.51	0.85	4296	5138	8.36	9.14
2345	" ⅝	"	"	"	"	64.01	0.82	4381	5328	8.27	9.12
2346	" 11/16	"	"	"	"	66.51	0.79	4466	5516	8.19	9.11
2347	" ¾	"	"	"	"	69.01	0.76	4550	5701	8.12	9.09
*2348	20x½	26x⅝	3½x3½x⅜	5x3½x11/16	3½x3½x11/16	60.63	0.63	4358	5107	8.48	9.18
2349	" 9/16	"	"	"	"	63.13	0.60	4442	5299	8.39	9.17
2350	" ⅝	"	"	"	"	65.63	0.58	4527	5489	8.31	9.15
2351	" 11/16	"	"	"	"	68.13	0.56	4611	5675	8.23	9.13
2352	" ¾	"	"	"	"	70.63	0.54	4694	5860	8.15	9.11
*2353	20x½	26x⅝	3½x3½x⅜	5x3½x¾	3½x3½x¾	62.21	0.40	4489	5267	8.50	9.20
2354	" 9/16	"	"	"	"	64.71	0.38	4573	5459	8.41	9.19
2355	" ⅝	"	"	"	"	67.21	0.37	4657	5648	8.32	9.17
2356	" 11/16	"	"	"	"	69.71	0.35	4740	5834	8.25	9.15
2357	" ¾	"	"	"	"	72.21	0.34	4824	6017	8.17	9.13
22" × 26" Section. A Series.											
*2358	22x½	26x9/16	4x4x½	4x4x½	4x4x½	59.13	1.55	4811	4499	9.02	8.73
*2359	" 9/16	"	"	"	"	61.88	1.48	4928	4691	8.92	8.71
2360	" ⅝	"	"	"	"	64.63	1.41	5045	4879	8.83	8.69
2361	" 11/16	"	"	"	"	67.38	1.35	5163	5066	8.75	8.67
2362	" ¾	"	"	"	"	70.13	1.30	5282	5246	8.68	8.65

*Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

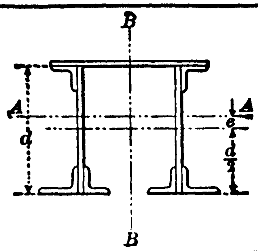
Section Number.	Plates.		Angles.			Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.				Axis A-A.	Axis B-B.	Axis A-A.	Axis B-B.
				Outside.	Inside.	A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴	Inches⁴	Inches.	Inches.
*2363	22x½	26x$\frac{9}{16}$	4x4x½	4x4x$\frac{9}{16}$	4x4x$\frac{9}{16}$	60.85	1.23	5023	4640	9.09	8.73
*2364	" $\frac{9}{16}$	"	"	"	"	63.60	1.18	5137	4832	8.99	8.71
2365	" ⅝	"	"	"	"	66.35	1.13	5252	5019	8.90	8.69
2366	" $\frac{11}{16}$	"	"	"	"	69.10	1.08	5367	5204	8.81	8.67
2367	" ¾	"	"	"	"	71.85	1.04	5483	5385	8.73	8.65
*2368	22x½	26x$\frac{9}{16}$	4x4x½	4x4x⅝	4x4x⅝	62.57	0.93	5219	4777	9.13	8.74
*2369	" $\frac{9}{16}$	"	"	"	"	65.32	0.89	5332	4967	9.03	8.72
2370	" ⅝	"	"	"	"	68.07	0.85	5445	5154	8.94	8.70
2371	" $\frac{11}{16}$	"	"	"	"	70.82	0.81	5558	5339	8.86	8.68
2372	" ¾	"	"	"	"	73.57	0.79	5671	5519	8.78	8.66
*2373	22x½	26x$\frac{9}{16}$	4x4x½	4x4x$\frac{11}{16}$	4x4x$\frac{11}{16}$	64.25	0.67	5397	4916	9.16	8.75
*2374	" $\frac{9}{16}$	"	"	"	"	67.00	0.64	5509	5106	9.06	8.73
2375	" ⅝	"	"	"	"	69.75	0.61	5620	5291	8.97	8.71
2376	" $\frac{11}{16}$	"	"	"	"	72.50	0.59	5732	5475	8.89	8.69
2377	" ¾	"	"	"	"	75.25	0.57	5844	5655	8.81	8.67
*2378	22x½	26x$\frac{9}{16}$	4x4x½	4x4x¾	4x4x¾	65.89	0.41	5563	5047	9.19	8.75
*2379	" $\frac{9}{16}$	"	"	"	"	68.64	0.40	5675	5235	9.09	8.73
2380	" ⅝	"	"	"	"	71.39	0.38	5786	5420	9.00	8.71
2381	" $\frac{11}{16}$	"	"	"	"	74.14	0.37	5888	5604	8.91	8.69
2382	" ¾	"	"	"	"	76.89	0.35	6009	5783	8.84	8.67
				22″ X 26″ Section. B Series.							
*2383	22x½	26x$\frac{9}{16}$	4x4x½	6x4x½	4x4x½	61.13	1.14	5104	4891	9.14	8.95
*2384	" $\frac{9}{16}$	"	"	"	"	63.88	1.09	5219	5083	9.04	8.93
2385	" ⅝	"	"	"	"	66.63	1.05	5333	5271	8.95	8.90
2386	" $\frac{11}{16}$	"	"	"	"	69.38	1.01	5446	5458	8.86	8.87
2387	" ¾	"	"	"	"	72.13	0.97	5560	5638	8.78	8.84
*2388	22x½	26x$\frac{9}{16}$	4x4x½	6x4x$\frac{9}{16}$	4x4x$\frac{9}{16}$	63.11	0.80	5333	5082	9.20	8.98
*2389	" $\frac{9}{16}$	"	"	"	"	65.86	0.77	5445	5274	9.10	8.95
2390	" ⅝	"	"	"	"	68.61	0.74	5557	5461	9.00	8.92
2391	" $\frac{11}{16}$	"	"	"	"	71.36	0.71	5670	5646	8.91	8.89
2392	" ¾	"	"	"	"	74.11	0.68	5782	5827	8.83	8.87

* Spacing of rivet lines of web greater than 30 X thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

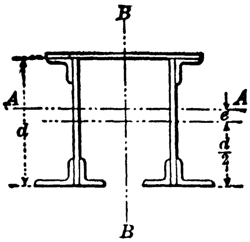
Section Number.	Plates.		Angles.			Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.				Axis A–A.	Axis B–B.	Axis A–A.	Axis B–B.
				Outside.	Inside.	A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2393	22x½	26x9/16	4x4x½	6x4x⅝	4x4x⅝	65.07	0.48	5544	5267	9.24	9.00
*2394	" 9/16	"	"	"	"	67.82	0.46	5656	5457	9.14	8.97
2395	" ⅝	"	"	"	"	70.57	0.44	5767	5644	9.04	8.94
2396	" 11/16	"	"	"	"	73.32	0 42	5879	5829	8.95	8.92
2397	" ¾	"	"	"	"	76.07	0.41	5991	6009	8.87	8.89
*2398	22x½	26x9/16	4x4x½	6x4x11/16	4x4x11/16	66.99	0.19	5735	5456	9.25	9.02
*2399	" 9/16	"	"	"	"	69.74	0.19	5846	5646	9.15	8.99
2400	" ⅝	"	"	"	"	72.49	0.18	5957	5831	9.06	8.97
2401	" 11/16	"	"	"	"	75.24	0.18	6068	6015	8.98	8.94
2402	" ¾	"	"	"	"	77.99	0.17	6179	6195	8.90	8.91
*2403	22x½	26x9/16	4x4x½	6x4x¾	4x4x¾	68.89	−.07	5913	5636	9.26	9.04
*2404	" 9/16	"	"	"	"	71.64	−.07	6024	5824	9.16	9.01
2405	" ⅝	"	"	"	"	74.39	−.07	6135	6009	9.08	8.98
2406	" 11/16	"	"	"	"	77.14	−.06	6246	6193	8.99	8.96
2407	" ¾	"	"	"	"	79.89	−.06	6357	6372	8.92	8.93
22″ × 26″ Section. C Series.											
*2408	22x½	26x9/16	4x4x½	6x4x½	6x4x½	63.13	0.77	5378	4915	9.23	8.82
*2409	" 9/16	"	"	"	"	65.88	0.73	5491	5106	9.13	8.80
2410	" ⅝	"	"	"	"	68.63	0.70	5604	5293	9.04	8.78
2411	" 11/16	"	"	"	"	71.38	0.67	5716	5479	8.95	8.76
2412	" ¾	"	"	"	"	74.13	0.65	5828	5659	8.86	8.73
*2413	22x½	26x9/16	4x4x½	6x4x9/16	6x4x9/16	65.37	0.40	5621	5110	9.28	8.84
*2414	" 9/16	"	"	"	"	68.12	0.38	5732	5301	9.17	8.82
2415	" ⅝	"	"	"	"	70.87	0.37	5844	5487	9.08	8.80
2416	" 11/16	"	"	"	"	73.62	0.36	5955	5671	8.99	8.78
2417	" ¾	"	"	"	"	76.37	0.35	6066	5851	8.92	8.76
*2418	22x½	26x9/16	4x4x½	6x4x⅝	6x4x⅝	67.57	0.07	5845	5298	9.31	8.86
*2419	" 9/16	"	"	"	"	70.32	0.07	5956	5487	9.21	8.84
2420	" ⅝	"	"	"	"	73.07	0.07	6067	5673	9.12	8.82
2421	" 11/16	"	"	"	"	75.82	0.06	6178	5857	9.03	8.80
2422	" ¾	"	"	"	"	78.57	0.06	6289	6035	8.95	8.77

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

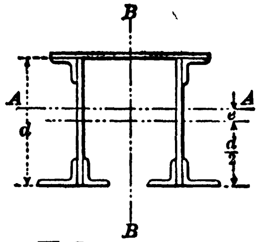
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom. Outside.	Angles. Bottom. Inside.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A–A. I_A	Moments of Inertia. Axis B–B. I_B	Radii of Gyration. Axis A–A. r_A	Radii of Gyration. Axis B–B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2423	22x1/2	26x9/16	4x4x1/2	6x4x11/16	6x4x11/16	69.73	−.23	6047	5480	9.32	8.87
*2424	" 9/16	"	"	"	"	72.48	−.22	6158	5679	9.21	8.85
2425	" 5/8	"	"	"	"	75.23	−.21	6269	5863	9.12	8.83
2426	" 11/16	"	"	"	"	77.98	−.20	6380	6046	9.04	8.80
2427	" 3/4	"	"	"	"	80.73	−.19	6491	6224	8.96	8.78
*2428	22x1/2	26x9/16	4x4x1/2	6x4x3/4	6x4x3/4	71.89	−.51	6233	5773	9.32	8.87
*2429	" 9/16	"	"	"	"	74.64	−.49	6344	5860	9.22	8.85
2430	" 5/8	"	"	"	"	77.39	−.48	6455	6044	9.14	8.83
2431	" 11/16	"	"	"	"	80.14	−.47	6567	6227	9.06	8.81
2432	" 3/4	"	"	"	"	82.89	−.46	6678	6404	8.98	8.79
					22″ × 28″ Section.						
2433	22x5/8	28x5/8	4x4x1/2	6x4x3/8	6x4x3/8	66.94	1.89	5326	6156	8.92	9.59
2434	" 11/16	"	"	"	"	69.69	1.81	5447	6389	8.84	9.58
2435	" 3/4	"	"	"	"	72.44	1.74	5566	6620	8.77	9.56
2436	22x5/8	28x5/8	4x4x1/2	6x4x7/16	6x4x7/16	69.22	1.50	5636	6391	9.02	9.61
2437	" 11/16	"	"	"	"	71.97	1.44	5753	6623	8.94	9.59
2438	" 3/4	"	"	"	"	74.72	1.39	5870	6853	8.87	9.58
2439	22x5/8	28x5/8	4x4x1/2	6x4x1/2	6x4x1/2	71.50	1.14	5920	6627	9.10	9.62
2440	" 11/16	"	"	"	"	74.25	1.10	6035	6858	9.01	9.61
2441	" 3/4	"	"	"	"	77.00	1.06	6149	7087	8.94	9.60
2442	22x5/8	28x5/8	4x4x1/2	6x4x9/16	6x4x9/16	73.74	0.81	6184	6858	9.16	9.64
2443	" 11/16	"	"	"	"	76.49	0.78	6297	7088	9.07	9.63
2444	" 3/4	"	"	"	"	79.24	0.75	6409	7315	8.99	9.61
2445	22x5/8	28x5/8	4x4x1/2	6x4x5/8	6x4x5/8	75.94	0.50	6422	7086	9.20	9.66
2446	" 11/16	"	"	"	"	78.69	0.48	6534	7315	9.11	9.64
2447	" 3/4	"	"	"	"	81.44	0.47	6645	7542	9.04	9.63
2448	22x5/8	28x5/8	4x4x1/2	6x4x11/16	6x4x11/16	78.10	0.22	6642	7311	9.22	9.68
2449	" 11/16	"	"	"	"	80.85	0.21	6753	7539	9.14	9.66
2450	" 3/4	"	"	"	"	83.60	0.21	6864	7765	9.06	9.64

*Spacing of rivet lines greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

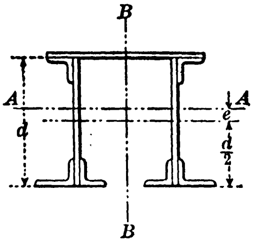
Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom. Outside.	Angles. Bottom. Inside.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches^2.	Inches.	Inches^4.	Inches^4.	Inches.	Inches.
2451	22x$\frac{5}{8}$	28x$\frac{5}{8}$	4x4x$\frac{1}{2}$	6x4x$\frac{3}{4}$	6x4x$\frac{3}{4}$	80.26	−.05	6851	7536	9.24	9.69
2452	" $\frac{11}{16}$	"	"	"	"	83.01	−.05	6962	7763	9.16	9.67
2453	" $\frac{3}{4}$	"	"	"	"	85.76	−.04	7073	7988	9.08	9.65
24″ × 28″ Section. A Series.											
*2454	24x$\frac{9}{16}$	28x$\frac{5}{8}$	4x4x$\frac{1}{2}$	4x4x$\frac{1}{2}$	4x4x$\frac{1}{2}$	67.00	2.00	6348	6117	9.73	9.56
*2455	" $\frac{5}{8}$	"	"	"	"	70.00	1.92	6502	6376	9.64	9.54
2456	" $\frac{11}{16}$	"	"	"	"	73.00	1.84	6656	6631	9.55	9.53
2457	" $\frac{3}{4}$	"	"	"	"	76.00	1.76	6810	6882	9.46	9.51
*2458	24x$\frac{9}{16}$	28x$\frac{5}{8}$	4x4x$\frac{1}{2}$	4x4x$\frac{9}{16}$	4x4x$\frac{9}{16}$	68.72	1.69	6617	6287	9.81	9.57
*2459	" $\frac{5}{8}$	"	"	"	"	71.72	1.62	6770	6545	9.72	9.55
2460	" $\frac{11}{16}$	"	"	"	"	74.72	1.56	6920	6799	9.63	9.54
2461	" $\frac{3}{4}$	"	"	"	"	77.72	1.50	7071	7050	9.54	9.52
*2462	24x$\frac{9}{16}$	28x$\frac{5}{8}$	4x4x$\frac{1}{2}$	4x4x$\frac{5}{8}$	4x4x$\frac{5}{8}$	70.44	1.38	6873	6456	9.88	9.58
*2463	" $\frac{5}{8}$	"	"	"	"	73.44	1.33	7021	6712	9.78	9.56
2464	" $\frac{11}{16}$	"	"	"	"	76.44	1.28	7170	6966	9.69	9.55
2465	" $\frac{3}{4}$	"	"	"	"	79.44	1.23	7319	7215	9.61	9.53
*2466	24x$\frac{9}{16}$	28x$\frac{5}{8}$	4x4x$\frac{1}{2}$	4x4x$\frac{11}{16}$	4x4x$\frac{11}{16}$	72.12	1.11	7103	6625	9.92	9.58
*2467	" $\frac{5}{8}$	"	"	"	"	75.12	1.07	7250	6880	9.82	9.56
2468	" $\frac{11}{16}$	"	"	"	"	78.12	1.03	7397	7133	9.72	9.55
2469	" $\frac{3}{4}$	"	"	"	"	81.12	1.00	7543	7382	9.63	9.53
*2470	24x$\frac{9}{16}$	28x$\frac{5}{8}$	4x4x$\frac{1}{2}$	4x4x$\frac{3}{4}$	4x4x$\frac{3}{4}$	73.76	0.86	7318	6785	9.96	9.59
*2471	" $\frac{5}{8}$	"	"	"	"	76.76	0.82	7465	7040	9.86	9.58
2472	" $\frac{11}{16}$	"	"	"	"	79.76	0.79	7611	7292	9.77	9.56
2473	" $\frac{3}{4}$	"	"	"	"	82.76	0.76	7767	7540	9.69	9.55
24″ × 28″ Section. B Series.											
*2474	24x$\frac{9}{16}$	28x$\frac{5}{8}$	4x4x$\frac{1}{2}$	6x4x$\frac{1}{2}$	4x4x$\frac{1}{2}$	69.00	1.61	6713	6567	9.87	9.76
*2475	" $\frac{5}{8}$	"	"	"	"	72.00	1.54	6865	6826	9.77	9.74
2476	" $\frac{11}{16}$	"	"	"	"	75.00	1.48	7015	7081	9.67	9.72
2477	" $\frac{3}{4}$	"	"	"	"	78.00	1.43	7164	7332	9.58	9.69

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

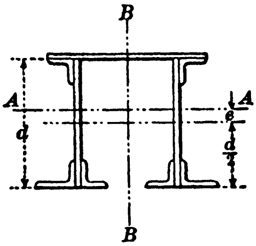
Section Number.	Plates.		Angles.			Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.				Axis A–A.	Axis B–B.	Axis A–A.	Axis B–B.
				Outside.	Inside.	A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2478	$24x\frac{9}{16}$	$28x\frac{5}{8}$	$4x4x\frac{1}{2}$	$6x4x\frac{9}{16}$	$4x4x\frac{9}{16}$	70.98	1.26	7010	6794	9.94	9.78
*2479	“ $\frac{5}{8}$	“	“	“	“	73.98	1.21	7158	7052	9.84	9.76
2480	“ $\frac{11}{16}$	“	“	“	“	76.98	1.17	7305	7306	9.74	9.74
2481	“ $\frac{3}{4}$	“	“	“	“	79.98	1.13	7452	7557	9.65	9.72
*2482	$24x\frac{9}{16}$	$28x\frac{5}{8}$	$4x4x\frac{1}{2}$	$6x4x\frac{5}{8}$	$4x4x\frac{5}{8}$	72.94	0.94	7285	7019	9.99	9.81
*2483	“ $\frac{5}{8}$	“	“	“	“	75.94	0.90	7431	7275	9.89	9.79
2484	“ $\frac{11}{16}$	“	“	“	“	78.94	0.87	7577	7529	9.80	9.77
2485	“ $\frac{3}{4}$	“	“	“	“	81.94	0.84	7723	7778	9.71	9.75
*2486	$24x\frac{9}{16}$	$28x\frac{5}{8}$	$4x4x\frac{1}{2}$	$6x4x\frac{11}{16}$	$4x4x\frac{11}{16}$	74.86	0.64	7535	7244	10.03	9.84
*2487	“ $\frac{5}{8}$	“	“	“	“	77.86	0.62	7680	7499	9.93	9.82
2488	“ $\frac{11}{16}$	“	“	“	“	80.86	0.60	7825	7752	9.84	9.80
2489	“ $\frac{3}{4}$	“	“	“	“	83.86	0.58	7970	8001	9.75	9.77
*2490	$24x\frac{9}{16}$	$28x\frac{5}{8}$	$4x4x\frac{1}{2}$	$6x4x\frac{3}{4}$	$4x4x\frac{3}{4}$	76.76	0.36	7770	7460	10.05	9.86
*2491	“ $\frac{5}{8}$	“	“	“	“	79.76	0.35	7913	7715	9.96	9.83
2492	“ $\frac{11}{16}$	“	“	“	“	82.76	0.34	8057	7967	9.87	9.81
2493	“ $\frac{3}{4}$	“	“	“	“	85.76	0.33	8202	8215	9.78	9.79
24″ × 28″ Section. C Series.											
*2494	$24x\frac{9}{16}$	$28x\frac{5}{8}$	$4x4x\frac{1}{2}$	$6x4x\frac{1}{2}$	$6x4x\frac{1}{2}$	71.00	1.23	7061	6606	9.98	9.65
*2495	“ $\frac{5}{8}$	“	“	“	“	74.00	1.19	7208	6864	9.87	9.63
2496	“ $\frac{11}{16}$	“	“	“	“	77.00	1.14	7356	7119	9.78	9.62
2497	“ $\frac{3}{4}$	“	“	“	“	80.00	1.10	7503	7368	9.69	9.60
*2498	$24x\frac{9}{16}$	$28x\frac{5}{8}$	$4x4x\frac{1}{2}$	$6x4x\frac{9}{16}$	$6x4x\frac{9}{16}$	73.24	0.85	7379	6838	10.04	9.66
*2499	“ $\frac{5}{8}$	“	“	“	“	76.24	0.82	7525	7095	9.93	9.64
2500	“ $\frac{11}{16}$	“	“	“	“	79.24	0.79	7671	7348	9.84	9.63
2501	“ $\frac{3}{4}$	“	“	“	“	82.24	0.76	7817	7598	9.75	9.61
*2502	$24x\frac{9}{16}$	$28x\frac{5}{8}$	$4x4x\frac{1}{2}$	$6x4x\frac{5}{8}$	$6x4x\frac{5}{8}$	75.44	0.53	7670	7068	10.08	9.68
*2503	“ $\frac{5}{8}$	“	“	“	“	78.44	0.51	7815	7322	9.98	9.67
2504	“ $\frac{11}{16}$	“	“	“	“	81.44	0.49	7960	7575	9.89	9.65
2505	“ $\frac{3}{4}$	“	“	“	“	84.44	0.47	8104	7823	9.80	9.63

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



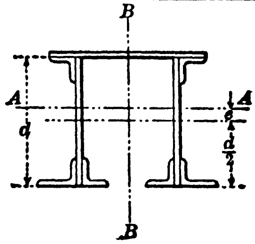
Six Angles and Three Plates.

Section Number.	Plates. Web.	Plates. Cover.	Angles. Top.	Angles. Bottom. Outside.	Angles. Bottom. Inside.	Gross Area. A	Eccentricity. e	Moments of Inertia. Axis A-A. I_A	Moments of Inertia. Axis B-B. I_B	Radii of Gyration. Axis A-A. r_A	Radii of Gyration. Axis B-B. r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2506	24x$\frac{9}{16}$	28x$\frac{5}{8}$	4x4x$\frac{1}{2}$	6x4x$\frac{11}{16}$	6x4x$\frac{11}{16}$	77.60	0.20	7937	7298	10.10	9.70
*2507	" $\frac{5}{8}$	"	"	"	"	80.60	0.20	8081	7551	10.00	9.68
2508	" $\frac{11}{16}$	"	"	"	"	83.60	0.19	8225	7803	9.92	9.66
2509	" $\frac{3}{4}$	"	"	"	"	86.60	0.19	8369	8051	9.83	9.64
*2510	24x$\frac{9}{16}$	28x$\frac{5}{8}$	4x4x$\frac{1}{2}$	6x4x$\frac{3}{4}$	6x4x$\frac{3}{4}$	79.76	—.08	8185	7519	10.12	9.71
*2511	" $\frac{5}{8}$	"	"	"	"	82.76	—.08	8329	7772	10.02	9.69
2512	" $\frac{11}{16}$	"	"	"	"	85.76	—.07	8473	8022	9.93	9.67
2513	" $\frac{3}{4}$	"	"	"	"	88.76	—.07	8617	8269	9.85	9.65
					24″ × 30″ Section.						
*2514	24x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{3}{8}$	6x4x$\frac{3}{8}$	72.57	2.43	6831	7875	9.70	10.42
2515	" $\frac{11}{16}$	"	"	"	"	75.57	2.33	6993	8187	9.62	10.41
2516	" $\frac{3}{4}$	"	"	"	"	78.57	2.24	7152	8498	9.53	10.40
*2517	24x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{7}{16}$	6x4x$\frac{7}{16}$	74.85	2.02	7228	8157	9.83	10.44
2518	" $\frac{11}{16}$	"	"	"	"	77.85	1.94	7384	8468	9.74	10.43
2519	" $\frac{3}{4}$	"	"	"	"	80.85	1.87	7539	8778	9.66	10.42
*2520	24x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{1}{2}$	6x4x$\frac{1}{2}$	77.13	1.64	7593	8439	9.92	10.46
2521	" $\frac{11}{16}$	"	"	"	"	80.13	1.59	7745	8749	9.84	10.45
2522	" $\frac{3}{4}$	"	"	"	"	83.13	1.52	7896	9057	9.75	10.44
*2523	24x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{9}{16}$	6x4x$\frac{9}{16}$	79.37	1.29	7934	8716	10.00	10.47
2524	" $\frac{11}{16}$	"	"	"	"	82.37	1.24	8083	9025	9.91	10.46
2525	" $\frac{3}{4}$	"	"	"	"	85.37	1.20	8231	9332	9.82	10.45
*2526	24x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{5}{8}$	6x4x$\frac{5}{8}$	81.57	0.96	8248	8989	10.05	10.50
2527	" $\frac{11}{16}$	"	"	"	"	84.57	0.93	8395	9297	9.97	10.48
2528	" $\frac{3}{4}$	"	"	"	"	87.57	0.90	8541	9603	9.88	10.46
*2529	24x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{11}{16}$	6x4x$\frac{11}{16}$	83.73	0.66	8531	9258	10.09	10.52
2530	" $\frac{11}{16}$	"	"	"	"	86.73	0.64	8677	9565	10.00	10.50
2531	" $\frac{3}{4}$	"	"	"	"	89.73	0.62	8822	9870	9.91	10.49

*Spacing of rivet lines of web greater than 30 × thickness of plate.

PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



Six Angles and Three Plates.

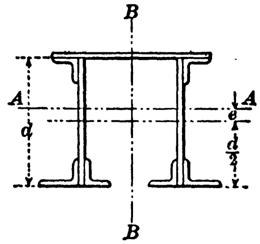
Section Number.	Plates.		Angles.			Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.				Axis A-A.	Axis B-B.	Axis A-A.	Axis B-B.
				Outside.	Inside.	A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches².	Inches.	Inches⁴.	Inches⁴.	Inches.	Inches.
*2532	24x5/8	30x11/16	4x4x1/2	6x4x3/4	6x4x3/4	85.89	0.38	8806	9526	10.13	10.53
2533	" 11/16	"	"	"	"	88.89	0.37	8950	9832	10.04	10.52
2534	" 3/4	"	"	"	"	91.89	0.36	9094	10135	9.95	10.50
26" × 30" Section. A Series.											
*2535	26x5/8	30x11/16	4x4x1/2	4x4x1/2	4x4x1/2	75.63	2.47	8220	8157	10.38	10.38
*2536	" 11/16	"	"	"	"	78.88	2.37	8421	8499	10.32	10.37
2537	" 3/4	"	"	"	"	82.13	2.27	8623	8834	10.26	10.36
*2538	26x5/8	30x11/16	4x4x1/2	4x4x9/16	4x4x9/16	77.35	2.15	8559	8363	10.52	10.40
*2539	" 11/16	"	"	"	"	80.60	2.06	8757	8704	10.43	10.39
2540	" 3/4	"	"	"	"	83.85	1.98	8953	9038	10.34	10.38
*2541	26x5/8	30x11/16	4x4x1/2	4x4x5/8	4x4x5/8	79.07	1.85	8878	8563	10.59	10.41
*2542	" 11/16	"	"	"	"	82.32	1.78	9062	8904	10.49	10.40
2543	" 3/4	"	"	"	"	85.57	1.71	9265	9237	10.40	10.39
*2544	26x5/8	30x11/16	4x4x1/2	4x4x11/16	4x4x11/16	80.75	1.57	9169	8764	10.65	10.42
*2545	" 11/16	"	"	"	"	84.00	1.51	9360	9103	10.55	10.41
2546	" 3/4	"	"	"	"	87.25	1.45	9551	9425	10.45	10.39
*2547	26x5/8	30x11/16	4x4x1/2	4x4x3/4	4x4x3/4	82.39	1.32	9441	8962	10.70	10.43
*2548	" 11/16	"	"	"	"	85.64	1.27	9629	9301	10.60	10.42
2549	" 3/4	"	"	"	"	88.89	1.22	9817	9632	10.50	10.41
26" × 30" Section. B Series.											
*2550	26x5/8	30x11/16	4x4x1/2	6x4x1/2	4x4x1/2	77.63	2.08	8669	8669	10.56	10.57
*2551	" 11/16	"	"	"	"	80.88	2.00	8865	9011	10.46	10.55
2552	" 3/4	"	"	"	"	84.13	1.92	9061	9346	10.37	10.53
*2553	26x5/8	30x11/16	4x4x1/2	6x4x9/16	4x4x9/16	79.61	1.73	9042	8939	10.65	10.60
*2554	" 11/16	"	"	"	"	82.86	1.65	9238	9280	10.55	10.58
2555	" 3/4	"	"	"	"	86.11	1.57	9434	9614	10.46	10.56
*2556	26x5/8	30x11/16	4x4x1/2	6x4x5/8	4x4x5/8	81.57	1.41	9389	9203	10.72	10.62
*2557	" 11/16	"	"	"	"	84.82	1.36	9577	9544	10.62	10.60
2558	" 3/4	"	"	"	"	88.07	1.31	9766	9877	10.53	10.58

* Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 85.—*Continued.*

Properties of Top Chord Sections.

Properties of Top Chord Sections.



Six Angles and Three Plates.

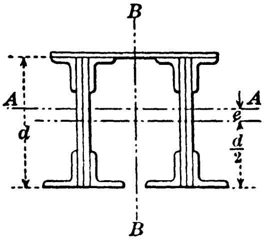
Section Number.	Plates.		Angles.			Gross Area.	Eccentricity.	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.	Bottom.				Axis A–A.	Axis B–B.	Axis A–A.	Axis B–B.
				Outside.	Inside.	A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches^2.	Inches.	Inches^4.	Inches^4.	Inches.	Inches.
*2559	26x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{11}{16}$	4x4x$\frac{11}{16}$	83.49	1.11	9707	9468	10.78	10.64
*2560	“ $\frac{11}{16}$	“	“	“	“	86.74	1.07	9894	9807	10.68	10.62
2561	“ $\frac{3}{4}$	“	“	“	“	89.99	1.03	10081	10139	10.58	10.61
*2562	26x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{3}{4}$	4x4x$\frac{3}{4}$	85.39	0.82	10011	9730	10.83	10.67
*2563	“ $\frac{11}{16}$	“	“	“	“	88.64	0.80	10195	10069	10.72	10.65
2564	“ $\frac{3}{4}$	“	“	“	“	91.89	0.78	10379	10400	10.62	10.63
26″ × 30″ Section. C Series.											
*2565	26x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{1}{2}$	6x4x$\frac{1}{2}$	79.63	1.70	9100	8727	10.69	10.46
*2566	“ $\frac{11}{16}$	“	“	“	“	82.88	1.63	9292	9067	10.59	10.45
2567	“ $\frac{3}{4}$.	“	“	“	“	86.13	1.57	9481	9403	10.49	10.44
*2568	26x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{9}{16}$	6x4x$\frac{9}{16}$	81.87	1.33	9500	9004	10.76	10.48
*2569	“ $\frac{11}{16}$	“	“	“	“	85.12	1.28	9688	9343	10.66	10.47
2570	“ $\frac{3}{4}$	“	“	“	“	88.37	1.24	9875	9676	10.56	10.46
*2571	26x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{5}{8}$	6x4x$\frac{5}{8}$	84.07	0.99	9870	9275	10.83	10.50
*2572	“ $\frac{11}{16}$	“	“	“	“	87.32	0.95	10056	9614	10.73	10.49
2573	“ $\frac{3}{4}$	“	“	“	“	90.57	0.91	10243	9946	10.63	10.47
*2574	26x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{11}{16}$	6x4x$\frac{11}{16}$	86.23	0.66	10212	9548	10.88	10.51
*2575	“ $\frac{11}{16}$	“	“	“	“	89.48	0.63	10397	9885	10.77	10.50
2576	“ $\frac{3}{4}$	“	“	“	“	92.73	0.61	10582	10215	10.67	10.49
*2577	26x$\frac{5}{8}$	30x$\frac{11}{16}$	4x4x$\frac{1}{2}$	6x4x$\frac{3}{4}$	6x4x$\frac{3}{4}$	88.39	0.37	10530	9817	10.92	10.53
*2578	“ $\frac{11}{16}$	“	“	“	“	91.64	0.36	10723	10154	10.82	10.52
2579	“ $\frac{3}{4}$	“	“	“	“	94.89	0.35	10897	10483	10.72	10.51
26″ × 32″ Section.											
2580	26x$\frac{3}{4}$	32x$\frac{3}{4}$	4x4x$\frac{1}{2}$	6x4x$\frac{3}{8}$	6x4x$\frac{3}{8}$	84.94	2.77	9017	10718	10.30	11.23
2581	“	“	“	“ $\frac{7}{16}$	“ $\frac{7}{16}$	87.22	2.39	9498	11048	10.44	11.25
2582	“	“	“	“ $\frac{1}{2}$	“ $\frac{1}{2}$	89.50	2.03	9948	11379	10.54	11.27
2583	“	“	“	“ $\frac{9}{16}$	“ $\frac{9}{16}$	91.74	1.69	10369	11703	10.63	11.29
2584	“	“	“	“ $\frac{5}{8}$	“ $\frac{5}{8}$	93.94	1.37	10761	12023	10.70	11.31
2585	“	“	“	“ $\frac{11}{16}$	“ $\frac{11}{16}$	96.10	1.06	11124	12338	10.76	11.33
2586	“	“	“	“ $\frac{3}{4}$	“ $\frac{3}{4}$	98.26	0.80	11466	12652	10.80	11.35

*Spacing of rivet lines of web greater than 30 × thickness of plate.

TABLE 86.

Properties of Top Chord Sections.

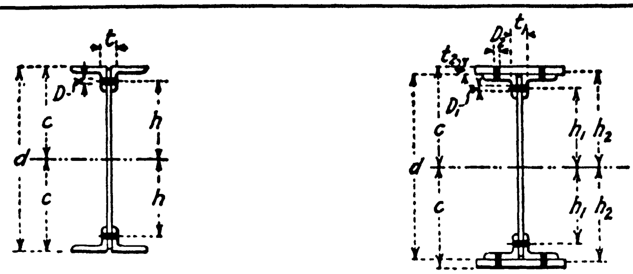
Properties of Extra Heavy Top Chord Sections.



Eight Angles with Short Legs Turned Out and Five Plates.

Section Number.	Plates.		Angles.				Gross Area.	Eccentricity	Moments of Inertia.		Radii of Gyration.	
	Web.	Cover.	Top.		Bottom.				Axis A–A.	Axis B–B.	Axis A–A.	Axis B–B.
			Outside.	Inside.	Outside.	Inside.	A	e	I_A	I_B	r_A	r_B
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.[2]	Inches.	Inches.[4]	Inches.[4]	Inches.	Inches
22″ × 28″ Section.												
2901	22×7/16	28×1/2	6×4×1/2	6×4×1/2	6×6×5/8	6×6×5/8	99.94	0.65	7436	9070	8.62	9.53
2902	" 1/2	"	"	"	"	"	105.44	0.62	7660	9478	8.52	9.48
2903	" 9/16	"	"	"	"	"	110.94	0.59	7884	9871	8.42	9.43
2904	" 5/8	"	"	"	"	"	116.44	0.56	8107	10255	8.34	9.38
2905	" 11/16	"	"	"	"	"	121.94	0.53	8330	10627	8.26	9.33
2906	" 3/4	"	"	"	"	"	127.44	0.51	8554	10987	8.19	9.28
24″ × 30″ Section.												
2907	24×1/2	30×5/8	6×4×1/2	6×4×1/2	6×6×3/4	6×6×3/4	119.51	0.64	10710	12874	9.47	10.38
2908	" 9/16	"	"	"	"	"	125.51	0.61	11000	13413	9.36	10.34
2909	" 5/8	"	"	"	"	"	131.51	0.58	11290	13934	9.27	10.29
2910	" 11/16	"	"	"	"	"	137.51	0.55	11580	14441	9.18	10.25
2911	" 3/4	"	"	"	"	"	143.51	0.53	11870	14937	9.10	10.20
26″ × 32″ Section.												
2912	26×9/16	32×5/8	6×4×1/2	6×4×1/2	6×6×3/4	6×6×3/4	131.26	0.74	13505	16638	10.14	11.26
2913	" 5/8	"	"	"	"	"	137.76	0.70	13874	17335	10.03	11.22
2914	" 11/16	"	"	"	"	"	144.26	0.67	14243	18015	9.94	11.17
2915	" 3/4	"	"	"	"	"	150.76	0.64	14613	18682	9.85	11.13
28″ × 34″ Section.												
2916	28×5/8	34×5/8	6×4×1/2	6×4×1/2	6×6×3/4	6×6×3/4	144.01	0.83	16791	21238	10.80	12.14
2917	" 11/16	"	"	"	"	"	151.01	0.79	17253	22126	10.69	12.10
2918	" 3/4	"	"	"	"	"	158.01	0.76	17715	22997	10.59	12.06
30″ × 36″ Section.												
2919	30×11/16	36×5/8	6×4×1/2	6×4×1/2	6×6×3/4	6×6×3/4	157.76	0.92	20627	26810	11.44	13.03
2920	" 3/4	"	"	"	"	"	165.26	0.88	21196	27920	11.33	13.00

TABLE 87.
Properties of Plate Girders.



Some specifications require that plate girders be proportioned by the moment of inertia of their gross section and some by the moment of inertia of their net section. The moment of inertia of the gross section can be obtained by direct addition from Tables 3, 5 and 33. The moment of inertia of the net section is obtained by subtracting the moment of inertia of the holes from that of the gross section. The moment of inertia of the holes can be calculated by the formula $I = A_0h^2$, the moment of inertia of the holes about their own axis being negligible, A_0 being the diametral **area** of the hole and h the distance from the neutral axis to the center of the hole.

The method of calculating the moments of inertia of plate girders will be illustrated by a typical example.

Example: Determine the moment of inertia and section modulus of a section consisting of 4 angles 5″x3½″x½″, long legs out, 24½″ back to back, 1 web plate 24″x⅜″, 2 cov. plates 12″x⅝″.

Moment of Inertia and Section Modulus of Gross Section.

Item.	b. to b. Angles.	Extreme Fiber.	Moment of Inertia, Axis A-A.		Section Modulus.
	d	c	Table.	I	S = I c.
Inches.	Inches.	Inches.		Inches⁴.	Inches³.
4 ∠ 5x3½x½ 1 Wb. Pl. 24x⅜ 2 Cov. Pl. 12x⅝	24.5 " "	12.25 + 0.625	33 3 5	2074 432 2366	$\frac{4872}{12.875}$
		12.875	Total I =	4872	S = 378.4

Moment of Inertia of Rivet Holes (⅞″ Rivets, 1″ holes).

Location.	Number.	Size.	Area.	Dist. to ¢ of Hole.	Dist.²	A_0h^2
		t × d	A_0 = t × d	h	h²	
		Inches.	Inches.²	Inches.	Inches².	Inches⁴.
Web	2	1⅜x1	2.75	10.3	106.1	292
Flange	4	1⅛x1	4.50	12.3	151.3	681
					Total =	973

The Moment of inertia of the net section is 4872 − 973 = 3899 in.⁴, and the section modulus is 3899 ÷ 12.875 = 302.8 in.³.

Approximate Methods.

The use of the moment of inertia of the net section in proportioning plate girders, requires that holes in the compression flange be deducted as well as those in the tension flange. This only approximates the true condition so that great accuracy in calculating the moment of inertia of the net section does not seem warranted. The following approximate solutions give results which are sufficiently accurate for use in design.

1st Approximate Method:

Net I of Angles = Gross $I \times \dfrac{\text{Net Area}}{\text{Gross Area}} = 2074 \times \dfrac{12}{16} =$ 1556 Table 33.
Net I of Web Pl. = Gross I of Net Depth = I of 22″ × ⅜″ Pl. = 333 " 3.
Net I of Cov. Pls. = Gross I of Net Width = I of 2 − 10″ × ⅝″ Pls. = 1972 " 5.
Total Moment of Inertia of Net Section = 3861 in.⁴

2d Approximate Method:

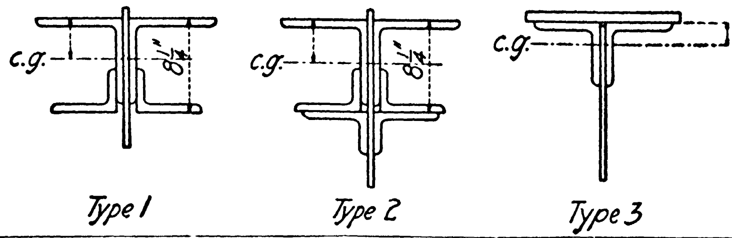
$$\text{Net } I = \text{Gross } I \times \frac{\text{Net Area}}{\text{Gross Area}} = 4872 \times \frac{32.75}{40.00} = 3989 \text{ in.}^4$$

This method gives more accurate results for sections without cover plates.

TABLE 88.

Centers of Gravity of Plate Girder Flanges.

CHICAGO, MILWAUKEE & ST. PAUL RY.



TYPE 1.						TYPE 2.						
Two Top Angles.	Two 6" x 4" Bottom Angles. Thickness in Inches.					Two Top Angles.	Four 6" x 4" Bottom Angles. Thickness in Inches.					
	⅜	½	⅝	¾	⅞		⅜	½	9/16	⅝	¾	⅞
Inches.	In.	In.	In.	In.	In.	Inches.	In.	In.	In.	In.	In.	In.
8×8×½	3.81	4.12	4.35	4.55	4.70	8×8×½	5.12	5.53	5.69	5.85	6.07	6.27
⅝	3.62	3.90	4.12	4.30	4.45	⅝	4.81	5.22	5.40	5.54	5.79	5.98
¾	3.49	3.75	3.96	4.13	4.27	¾	4.59	4.99	5.16	5.30	5.55	5.75
⅞	3.39	3.70	3.83	3.99	4.13	⅞	4.42	4.80	4.96	5.11	5.25	5.57
1	3.33	3.55	3.73	3.89	4.03	1	4.28	4.65	4.81	4.96	5.19	5.41
1⅛	3.28	3.48	3.67	3.81	3.94	1⅛	4.38	4.53	4.66	4.82	5.06	5.26

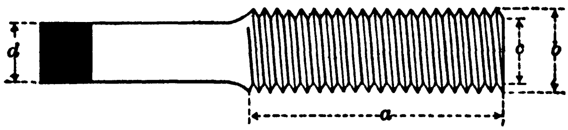
TYPE 3.

Size of Angles.	Width of Plate.	Thickness of Plate, Inches.																
In.	In.	0	⅜	½	⅝	¾	⅞	1	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2½	3
6×6×½	13	1.68	1.12	.98	.86	.73	.63	.52	.43	.33	.24	.15	.07	−.02	−.10	−.18		
	14		1.09	.95	.82	.70	.59	.48	.39	.29	.20	.11	.03	−.06	−.14	−.22		
	15		1.07	.92	.79	.66	.55	.45	.35	.25	.16	.07	−.01	−.10	−.18	−.26		
	16		1.04	.89	.75	.63	.52	.41	.31	.21	.12	.04	−.05	−.13	−.21	−.29		
6×6×⅝	13	1.73	1.24	1.11	.99	.87	.77	.67	.57	.47	.38	.30	.21	.13	.04	−.04		
	14		1.21	1.08	.95	.83	.73	.63	.53	.43	.34	.25	.17	.08	.00	−.08		
	15		1.19	1.05	.92	.80	.69	.59	.49	.39	.30	.21	.13	.04	−.04	−.12		
	16		1.16	1.02	.89	.77	.65	.55	.45	.35	.26	.17	.09	.00	−.08	−.16		
6×6×¾	13	1.78	1.34	1.21	1.10	.99	.89	.79	.69	.60	.51	.42	.34	.25	.16	.09		
	14		1.31	1.18	1.07	.95	.85	.75	.65	.55	.46	.38	.29	.20	.12	.05		
	15		1.29	1.15	1.03	.92	.81	.71	.61	.51	.42	.33	.25	.16	.06	.00		
	16		1.26	1.13	1.00	.88	.78	.67	.59	.47	.38	.29	.21	.12	.03	−.04		
6×6×⅞	13	1.82	1.42	1.30	1.19	1.09	.99	.89	.80	.71	.62	.54	.45	.37	.29	.21		
	14		1.39	1.27	1.16	1.05	.95	.85	.76	.66	.57	.49	.40	.32	.24	.16		
	15		1.37	1.24	1.13	1.01	.91	.81	.72	.62	.53	.44	.36	.27	.19	.11		
	16		1.35	1.22	1.10	.98	.87	.78	.68	.58	.49	.40	.32	.22	.14	.07		
8×8×½	17	2.19	1.48	1.32	1.17	1.03	.90	.78		.56		.36		.17		−.01	−.33	−.64
	18		1.46	1.29	1.14	1.00	.86	.74		.52		.32		.13		−.04	−.37	−.68
8×8×⅝	17	2.23	1.63	1.47	1.32	1.19	1.07	.95		.73		.53		.34		.17	−.16	−.48
	18		1.60	1.44	1.29	1.15	1.02	.91		.69		.49		.30		.12	−.21	−.52
8×8×¾	17	2.28	1.75	1.60	1.46	1.33	1.22	1.10		.88		.68		.46		.31	−.02	−.36
	18		1.72	1.57	1.43	1.29	1.17	1.06		.84		.64		.42		.27	−.06	−.40
8×8×⅞	17	2.32	1.85	1.71	1.57	1.45	1.33	1.22		1.00		.81		.62		.45	.11	−.20
	18		1.81	1.67	1.53	1.41	1.29	1.18		.96		.77		.57		.40	.07	−.25
8×8×1	17	2.37	1.94	1.80	1.68	1.55	1.45	1.35		1.13		.94		.75		.58	.25	−.08
	18		1.90	1.76	1.64	1.51	1.40	1.30		1.09		.89		.71		.53	.20	−.12
8×8×1⅛	17	2.41	2.02	1.89	1.77	1.66	1.55	1.45		1.25		1.05		.87		.70	.36	.06
	18		1.98	1.85	1.73	1.62	1.50	1.40		1.20		1.00		.83		.65	.32	.01

TABLE 89.

Upset Screw Ends for Square Bars.

AMERICAN BRIDGE COMPANY STANDARD.



Pitch and Shape of Thread A. B. Co. Standard.

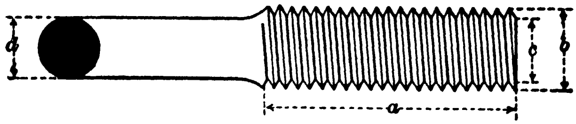
BAR.			UPSET.					
							Area.	
Side of Square d, Inches.	Area, Sq. Inches.	Weight per Foot, Lbs.	Diameter b, Inches.	Length a, Inches.	Additional Length for Upset +10%, Inches.	Diameter at Root of Thread c, Inches.	At Root of Thread, Sq. Inches.	Excess Over Area of Bar, %.
* 3/4	0.563	1.91	1 1/8	4	4	0.939	0.693	23.2
* 7/8	0.766	2.60	1 1/4	4	3 1/2	1.064	0.890	16.2
1	1.000	3.40	1 1/2	4	4	1.283	1.294	29.4
1 1/8	1.266	4.30	1 5/8	4	3 1/2	1.389	1.515	19.7
1 1/4	1.563	5.31	1 7/8	4 1/2	4 1/2	1.615	2.049	31.1
1 3/8	1.891	6.43	2	4 1/2	4	1.711	2.300	21.7
1 1/2	2.250	7.65	2 1/4	5	5	1.961	3.021	34.3
1 5/8	2.641	8.98	2 3/8	5	4 1/2	2.086	3.419	29.5
1 3/4	3.063	10.41	2 1/2	5 1/2	4 1/2	2.175	3.716	21.3
1 7/8	3.516	11.95	2 3/4	5 1/2	5	2.425	4.619	31.4
2	4.000	13.60	2 7/8	6	5	2.550	5.108	27.7
2 1/8	4.516	15.35	3	6	4 1/2	2.629	5.428	20.2
2 1/4	5.063	17.21	3 1/4	6 1/2	5 1/2	2.879	6.509	28.6
2 3/8	5.641	19.18	3 1/2	7	6 1/2	3.100	7.549	33.8
2 1/2	6.250	21.25	3 3/4	7	7	3.317	8.641	38.3
2 5/8	6.891	23.43	3 3/4	7	5 1/2	3.317	8.641	25.4
2 3/4	7.563	25.71	4	7 1/2	6 1/2	3.567	9.993	32.1
2 7/8	8.266	28.10	4 1/4	8	7 1/2	3.798	11.330	37.1
3	9.000	30.60	4 1/4	8	6	3.798	11.330	25.9
3 1/8	9.766	33.20	4 1/2	8 1/2	7	4.028	12.741	30.5
3 1/4	10.563	35.91	4 3/4	8 1/2	7 1/2	4.255	14.221	34.6

Upsets marked * are special.

TABLE 90.

Upset Screw Ends for Round Bars.

AMERICAN BRIDGE COMPANY STANDARD.



Pitch and Shape of Thread A. B. Co. Standard.

BAR.			UPSET.					
Diameter d, Inches.	Area, Sq. Inches.	Weight per Foot, Lb.	Diameter b, Inches.	Length a, Inches.	Additional Length for Upset +10 %, Inches.	Diameter at Root of Thread c, Inches.	Area. At Root of Thread, Sq. Inches.	Area. Excess Over Area of Bar, %.
* 3/4	0.442	1.50	1	4	4	0.838	0.551	24.7
* 7/8	0.601	2.04	1 1/4	4	5	1.064	0.890	48.0
1	0.785	2.67	1 3/8	4	4	1.158	1.054	34.2
1 1/8	0.994	3.38	1 1/2	4	4	1.283	1.294	30.2
1 1/4	1.227	4.17	1 5/8	4	4	1.389	1.515	23.5
1 3/8	1.485	5.05	1 3/4	4	4	1.490	1.744	17.5
1 1/2	1.767	6.01	2	4 1/2	4 1/2	1.711	2.300	30.2
1 5/8	2.074	7.05	2 1/8	4 1/2	4	1.836	2.649	27.7
1 3/4	2.405	8.18	2 1/4	5	4	1.961	3.021	25.6
1 7/8	2.761	9.39	2 3/8	5	4	2.086	3.419	23.8
2	3.142	10.68	2 1/2	5 1/2	4	2.175	3.716	18.3
2 1/8	3.547	12.06	2 5/8	5 1/2	3 1/2	2.300	4.156	17.2
2 1/4	3.976	13.52	2 7/8	6	4 1/2	2.550	5.108	28.4
2 3/8	4.430	15.06	3	6	4 1/2	2.629	5.428	22.5
2 1/2	4.909	16.69	3 1/4	6 1/2	5 1/2	2.879	6.509	32.6
2 5/8	5.412	18.40	3 1/4	6 1/2	4 1/2	2.879	6.509	20.3
2 3/4	5.940	20.19	3 1/2	7	5 1/2	3.100	7.549	27.1
2 7/8	6.492	22.07	3 3/4	7	6	3.317	8.641	33.1
3	7.069	24.03	3 3/4	7	5	3.317	8.641	22.2
3 1/8	7.670	26.08	4	7 1/2	6	3.567	9.993	30.3
3 1/4	8.296	28.21	4	7 1/2	5	3.567	9.993	20.5
3 3/8	8.946	30.42	4 1/4	8	5 1/2	3.798	11.330	26.6
3 1/2	9.621	32.71	4 1/4	8	5	3.798	11.330	17.8
3 5/8	10.321	35.09	4 1/2	8 1/2	5 1/2	4.028	12.741	23.4
3 3/4	11.045	37.55	4 3/4	8 1/2	6	4.255	14.221	28.8
3 7/8	11.793	40.10	4 3/4	8 1/2	5 1/2	4.255	14.221	20.6

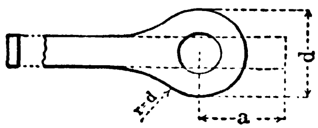
Upsets marked * are special.

TABLE 91.

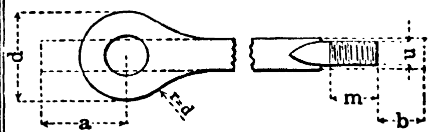
STANDARD EYE BARS.

AMERICAN BRIDGE COMPANY STANDARDS.

ORDINARY EYE BAR



ADJUSTABLE EYE BAR



Minimum length of short end from center of pin to end of screw, 6′-6″, preferably 7′-0″.

Thread on short end to be left hand.

Pitch and Shape of Thread A. B. Co. Standard.

Ordinary Eye Bar

BAR			HEAD				
Width In.	Thickness Max. In.	Thickness Min. In.	Dia. d, In.	Maximum Pin Dia. In.	Maximum Pin Excess Head over Bar, %	Additional Material, a, Ft. and In. For ordering Bar	Additional Material, a, Ft. and In. For figuring Weight
2	1	½	4½	1¾	37.5	0-10½	0- 7
			5½	2¾		1- 2½	0-11
			* 6½	3¾		1- 7½	1- 4
2½	1	⅝	6	2½	40.0	1- 1¾	0-10
			7	3½		1- 5¾	1- 2
			* 8	4½		1-10¾	1- 7
3	1½	⅝	7½	3¼	41.7	1- 4½	1- 1
			8½	4¼		1- 9½	1- 5
			* 9½	5¼		2- 2½	1-10
4	1¾	¾	10	4½	37.5	1- 9	1- 6
		⅞	11	5½		2- 3	1-10
		1	*12	6½		2- 8	2- 2
5	2	¾	12	5¼	35.0	1-10½	1- 8
		1	13½	6¾		2- 6	2- 2
		1	*15	8¼		3- 3	2- 9
6	2	¾	14	5¾	37.5	2- 1	1-10
		1	14¾	6½		2- 4	2- 1
		1	*16½	8¼		3- 2	2- 8
7	2	1	16½	7	35.7	2- 4½	2- 2
		1⅛	17½	8		2-11	2- 6
		1⅛	*18½	9		3- 4	2-11
8	2	1	18	7	37.5	2- 5½	2- 3
		1⅛	19	8		2- 9½	2- 6
		1¼	*20	9		3- 4	2-11
9	2	1⅛	20	7½	38.9	2- 8½	2- 6
		1¼	22	9½		3- 4½	3- 1
10	2	1⅛	22½	9	35.0	3- 2½	2-10
		1¼	24	10½		3- 9	3- 3
		1⅜	*25	11½		4- 1.	3- 7
12	2	1¼	26½	10	37.5	3- 4	3- 3
		1⅜	28	11½		4- 2	3- 8
		1½	*29½	13		4- 8	4- 1
14	2	1⅜	31	12	35.7	3-11	3- 9
		1½	†33	14		4- 7	4- 4
		1⅝	*34	15		5- 5	4- 8
16	2	1¾	36	14	37.5	4- 7	4- 5
		1⅞	*37½	16	34.4	4-11	4-10

Adjustable Eye Bar

BAR		SCREW END				
Width In.	Min. thickness In.	Dia. u, In.	Excess Upset over Bar %	Length m, In.	Additional Material, b, Ft. and In. For ordering Bar	Additional Material, b, Ft. and In. For figuring Wt.
2	* ⅝	1¾	39.6	4	1- 0	8
	¾	1⅞	36.6	4½	1- 0	7½
	⅞	2	31.4	4½	0-11	7½
2½	* ¾	2⅛	41.2	4½	1- 0	8
	⅞	2¼	38.1	5	1- 0	8
	1	2⅜	36.7	5	1- 0	7½
3	* ¾	2¼	34.3	5	1- 0	7½
	⅞	2½	41.6	5½	1 -1	9½
	1	2½	23.9	5½	1- 1	8½
4	* ¾	2½	23.9	5½	1- 1	8½
	⅞	2¾	32.0	5½	0-11	7½
	1	3	35.7	6	1- 1	8½
	1⅛	3¼	44.6	6½	1- 2	9½
5	* ¾	2⅞	36.2	6	1- 0	8
	⅞	3	24.1	6	0-11	7
	1	3¼	30.2	6½	1- 0	8
	1⅛	3½	34.2	7	1- 1	8½
	1¼	3¾	38.3	7	1- 2	9
6	*1	3½	25.8	7	1- 0	7½
	1⅛	3¾	28.0	7	1- 0	8
	1¼	4	33.2	7½	1- 1	8½
	1⅜	4¼	37.3	8	1- 2	9½
7	*1⅛	4	26.9	7½	1- 0	8
	1¼	4¼	29.5	8	1- 1	8½
	1⅜	4½	32.4	8½	1- 2	9
	1½	4¾	35.4	8½	1- 2	9½
8	*1⅛	4¼	25.9	8	1- 0	8
	1¼	4½	27.4	8½	1- 1	8½
	1⅜	4¾	29.3	8½	1- 1	8½
	1½	5	31.4	9	1- 2	9
	1⅝	5¼	35.2	9½	1- 3	10

Bars marked * should only be used when absolutely unavoidable.

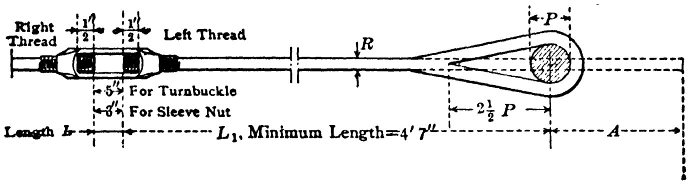
Deduct pin hole when figuring weight.

†For 14″ Bars, 33″ Head, over 1¾″ thick add 4′-5½″.

TABLE 92.

Loop Rods.

AMERICAN BRIDGE COMPANY STANDARD.



Pitch and Shape of Thread A. B. Co. Standard.

Additional Length "A" in Feet and Inches for One Loop.

$A = 4.17P + 5.89R.$

Diam. of Pin, P.	Diameter or Side "R" of Rod in Inches.										
	¾	⅞	1	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2
1⅛	0-9½	0-10	0-11	0-11½	......	......	......	......	......	......	......
1¼	0-10	0-10½	0-11½	1-0	1-1	......	......	......	......	......	......
1½	0-11	0-11½	1-0½	1-1	1-2	1-2½	......	......	......	......	......
1¾	1-0	1-0½	1-1½	1-2	1-3	1-3½	1-4½	1-5	1-6	......	......
2	1-1	1-1½	1-2½	1-3	1-4	1-4½	1-5½	1-6	1-7	1-7½	1-8½
2¼	1-2	1-3	1-3½	1-4½	1-5	1-5½	1-6½	1-7	1-8	1-8½	1-9½
2½	1-3	1-4	1-4½	1-5½	1-6	1-7	1-7½	1-8	1-9	1-9½	1-10½
2¾	1-4	1-5	1-5½	1-6½	1-7	1-8	1-8½	1-9½	1-10	1-11	1-11½
3	1-5	1-6	1-6½	1-7½	1-8	1-9	1-9½	1-10½	1-11	2-0	2-0½
*3¼	1-6	1-7	1-7½	1-8½	1-9	1-10	1-10½	1-11½	2-0	2-1	2-1½
3½	1-7½	1-8	1-8½	1-9½	1-10	1-11	1-11½	2-0½	2-1	2-2	2-2½
*3¾	1-8½	1-9	1-10	1-10½	1-11	2-0	2-0½	2-1½	2-2	2-3	2-3½
4	1-9½	1-10	1-11	1-11½	2-0½	2-1	2-2	2-2½	2-3	2-4	2-4½
*4¼	......	1-11	2-0	2-0½	2-1½	2-2	2-3	2-3½	2-4½	2-5	2-6
4½	......	2-0	2-1	2-1½	2-2½	2-3	2-4	2-4½	2-5½	2-6	2-7
*4¾	......	2-1	2-2	2-2½	2-3½	2-4	2-5	2-5½	2-6½	2-7	2-8
5	......	2-2½	2-3	2-3½	2-4½	2-5	2-6	2-6½	2-7½	2-8	2-9
*5¼	......	......	2-4	2-5	2-5½	2-6	2-7	2-7½	2-8½	2-9	2-10
5½	......	......	2-5	2-6	2-6½	2-7½	2-8	2-9	2-9½	2-10	2-11
*5¾	......	......	2-6	2-7	2-7½	2-8½	2-9	2-10	2-10½	2-11½	3-0
6	......	......	2-7	2-8	2-8½	2-9½	2-10	2-11	2-11½	3-0½	3-1
*6¼	......	......	......	2-9	2-9½	2-10½	2-11	3-0	3-0½	3-1½	3-2
6½	......	......	......	2-10	2-10½	2-11½	3-0	3-1	3-1½	3-2½	3-3
*6¾	......	......	......	2-11	3-0	3-0½	3-1	3-2	3-2½	3-3½	3-4
7	......	......	......	3-0	3-1	3-1½	3-2½	3-3	3-3½	3-4½	3-5

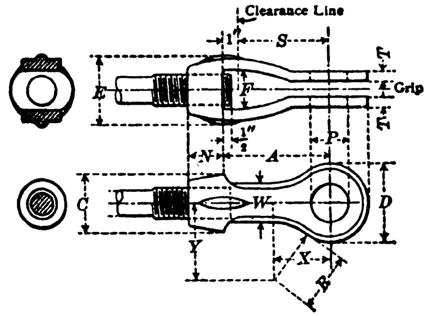
Pins marked * are special. Maximum shipping length of "L" = 35 feet.

TABLE 93.

Clevises.

AMERICAN BRIDGE COMPANY STANDARD.

All dimensions in inches.



Grip = thickness of plate + ⅛″.

Number of Clevis.	Head. Dia. D	Head. Thickness. T	Diameter of Pin, P. Max.	Diameter of Pin, P. Min.	Width. W	Extreme. E	Fork. F	Distance. A	Diameter of Upset. Max.	Diameter of Upset. Min.	Nut. N	Nut. B	Weight, Pounds.	Number of Clevis.
3	3	½	1½	1	1½	3$\frac{1}{16}$	1¼	5	1⅛	1	1½	2¼	4	3
4	4	½	2	1¼	2	3⅝	1¾	6	1⅝	1⅛	1¾	2⅞	8	4
5	5	⅝	2½	1½	2½	4½	2¼	7	2⅛	1½	2¼	3¾	16	5
6	6	¾	3	2	3	5⅜	2¾	8	2½	2	2½	4⅜	26	6
7	7	⅞	3½	2½	3½	6$\frac{3}{16}$	3¼	9	2⅞	2¼	3	5	36	7

Clevis Numbers for Various Rods and Pins.

Rods. Round.	Rods. Square.	Rods. Upset.	Pins. 1	1¼	1½	1¾	2	2¼	2½	2¾	3	3¼	3½
¾	......	1	3	3	3	......	......	......	......	......	......	......	......
⅞	¾	1⅛	3	3	3	4	4	......	......	......	......	......	......
......	⅞	1¼	......	4	4	4	4	......	......	......	......	......	......
1	......	1⅜	......	4	4	4	4	......	......	......	......	......	......
1⅛	1	1½	......	4	4	4	4	5	5	......	......	......	......
1¼	1⅛	1⅝	......	4	4	4	4	5	5	......	......	......	......
1⅜	......	1¾	......	......	5	5	5	5	5	......	......	......	......
1½	1¼	1⅞	......	......	5	5	5	5	5	......	......	......	......
1⅝	1⅜	2	......	......	5	5	5	5	5	6	6	......	......
1¾	......	2⅛	......	......	5	5	5	5	5	6	6	......	......
1⅞	1½	2¼	......	......	......	......	6	6	6	6	6	7	7
2	1⅝	2⅜	......	......	......	......	6	6	6	6	6	7	7
2⅛	1¾	2½	......	......	......	......	6	6	6	6	6	7	7
2¼	1⅞	2¾	......	......	......	......	......	......	7	7	7	7	7
2⅜	2	2⅞	......	......	......	......	......	......	7	7	7	7	7

Clevises to be used with the Rods and Pins given above.

Clevises above and to right of zigzag line may be used with forks straight, those below and to left of this line should have forks closed so as not to overstress pin.

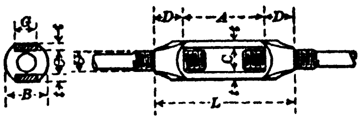
TABLE 94.

Turnbuckles and Sleeve Nuts.

American Bridge Company Standard.

All Dimensions in Inches.

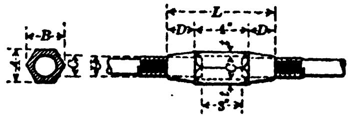
TURNBUCKLES.



A = 6"; A = 9"9 for turnbuckles marked *.
Pitch and shape of thread, A. B. Co. Standard.

Diam. of Screw. U	Standard Dimensions. D	L	C	t	G	B	Weight, Pounds.
3/8	9/16	7 1/8	9/16	3/16	1/2	1 1/16	1
7/16	21/32	7 5/16	5/8	1/4	5/8	1 3/8	1
1/2	3/4	7 1/2	5/8	1/4	5/8	1 3/8	1
9/16	27/32	7 11/16	13/16	5/16	3/4	1 9/16	1 1/2
5/8	15/16	7 7/8	13/16	5/16	3/4	1 9/16	1 1/2
3/4	1 1/8	8 1/4	1 1/16	11/32	7/8	2	2
7/8	1 5/16	8 5/8	1 1/4	3/8	1	2 1/4	3
1	1 1/2	9	1 5/16	7/16	1 1/4	2 7/16	4
1 1/8	1 11/16	9 3/8	1 7/16	1/2	1 1/4	2 9/16	5
1 1/4	1 7/8	9 3/4	1 9/16	1/2	1 1/2	2 3/4	6
1 3/8	2 1/16	10 1/8	1 11/16	1/2	1 5/8	3 1/16	7
1 1/2	2 1/4	10 1/2	1 3/4	5/8	1 3/4	3 3/16	8
1 5/8	2 7/16	10 7/8	2	5/8	1 7/8	3 1/2	10
1 3/4	2 5/8	11 1/4	2 1/8	5/8	2	3 3/4	11
1 7/8	2 13/16	11 5/8	2 3/16	11/16	2 1/8	3 7/8	12
2	3	12	2 3/8	11/16	2 1/4	4 1/4	14
2 1/8	3 3/16	12 3/8	2 1/2	23/32	2 1/2	4 1/2	17
2 1/4	3 3/8	12 3/4	2 11/16	13/16	2 1/2	4 3/4	20
2 3/8	3 9/16	13 1/8	2 3/4	13/16	2 3/4	4 7/8	22
2 1/2	3 3/4	13 1/2	3 1/16	27/32	3	5 3/8	25
2 3/4	4 1/8	14 1/4	3 1/4	15/16	3 1/4	5 3/4	33
2 7/8	4 5/16	14 5/8	3 7/16	1 1/32	3 1/4	6 1/16	36
3	4 1/2	15	3 5/8	1 1/32	3 1/2	6 3/8	40
3 1/4	4 7/8	15 3/4	3 7/8	1 1/16	4	6 3/4	50
3 1/2	5 1/4	16 1/2	4 1/4	1 7/32	4	7 1/4	65
3 3/4	5 5/8	17 1/4	4 7/16	1 5/16	5	8 1/4	95
4	6	18	4 5/8	1 7/16	5	8 3/4	108
*4 1/4	6 1/4	21 1/2	4 5/8	1 5/8	5 5/32	9 1/4	140
*4 1/2	6 3/4	22 1/2	5 1/2	1 3/4	6 1/2	10 3/4	195
*4 3/4	7 1/4	23 1/2	5 5/8	2	6 1/2	11 1/4	205
*5	7 1/2	24	6	2 1/4	6 1/2	11 7/8	250

SLEEVE NUTS.



Pitch and shape of thread, A. B. Co. Standard.

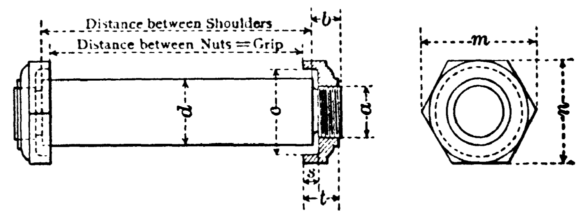
Diam. of Screw. U	Standard Dimensions. D	L	A	B	C	t	Weight, Pounds.
7/8	1 1/2	7	1 5/8	1 7/8	1 1/8	1/4	3
1	1 1/2	7	1 5/8	1 7/8	1 1/8	1/4	3
1 1/8	1 3/4	7 1/2	2	2 5/16	1 3/8	5/16	4
1 1/4	1 3/4	7 1/2	2	2 5/16	1 3/8	5/16	4
1 3/8	2	8	2 3/8	2 3/4	1 5/8	3/8	5
1 1/2	2	8	2 3/8	2 3/4	1 5/8	3/8	6
1 5/8	2 1/4	8 1/2	2 3/4	3 3/16	1 7/8	7/16	8
1 3/4	2 1/4	8 1/2	2 3/4	3 3/16	1 7/8	7/16	9
1 7/8	2 1/2	9	3 1/8	3 5/8	2 1/8	1/2	10
2	2 1/2	9	3 1/8	3 5/8	2 1/8	1/2	11
2 1/8	2 3/4	9 1/2	3 1/2	4 1/16	2 3/8	9/16	14
2 1/4	2 3/4	9 1/2	3 1/2	4 1/16	2 3/8	9/16	15
2 3/8	3	10	3 7/8	4 1/2	2 5/8	5/8	18
2 1/2	3	10	3 7/8	4 1/2	2 5/8	5/8	19
2 3/4	3 1/4	10 1/2	4 1/4	4 15/16	2 7/8	11/16	23
2 7/8	3 1/2	11	4 5/8	5 3/8	3 1/8	3/4	27
3	3 1/2	11	4 5/8	5 3/8	3 1/8	3/4	28
3 1/4	3 3/4	11 1/2	5	5 13/16	3 3/8	13/16	35
3 1/2	4	12	5 3/8	6 1/4	3 5/8	7/8	40
3 3/4	4 1/4	12 1/2	5 3/4	6 11/16	3 7/8	15/16	47
4	4 1/2	13	6 1/8	7 1/16	4 1/8	1	55
4 1/4	4 3/4	13 1/2	6 1/2	7 1/2	4 3/8	1 1/16	65
4 1/2	5	14	6 7/8	7 15/16	4 3/4	1 1/16	75

TABLE 95.

Bridge Pins and Nuts.

AMERICAN BRIDGE COMPANY STANDARD.

All Dimensions in Inches.



To obtain grip, add $\frac{1}{16}''$ for each bar. Nuts threaded 6 threads per inch.
To obtain distance between shoulders, add amount given in table to grip.

Diameter of Pin, d.			Pin. Thread. a	b	Add to Grip.	Nut. Thickness. t	Diameter. n	m	c	Depth s	Diameter Rough Hole.	Weight, Pounds.	Pattern No.
	2,	$2\frac{1}{4}$	$1\frac{1}{2}$	1	$\frac{1}{4}$	$\frac{7}{8}$	$2\frac{15}{16}$	$3\frac{3}{8}$	$2\frac{5}{8}$	$\frac{1}{4}$	$1\frac{5}{16}$	1.1	PN 21
	$2\frac{1}{2}$,	$2\frac{3}{4}$	2	$1\frac{1}{8}$	$\frac{1}{4}$	1	$3\frac{9}{16}$	$4\frac{1}{8}$	$3\frac{1}{8}$	$\frac{1}{4}$	$1\frac{13}{16}$	1.7	PN 22
3,	$*3\frac{1}{4}$,	$3\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{4}$	$\frac{1}{4}$	$1\frac{1}{8}$	$4\frac{5}{16}$	5	$3\frac{7}{8}$	$\frac{3}{8}$	$2\frac{5}{16}$	2.5	PN 23
	$*3\frac{3}{4}$,	4	3	$1\frac{3}{8}$	$\frac{1}{2}$	$1\frac{1}{4}$	$4\frac{7}{8}$	$5\frac{5}{8}$	$4\frac{3}{8}$	$\frac{3}{8}$	$2\frac{13}{16}$	3.7	PN 24
$*4\frac{1}{4}$,	$4\frac{1}{2}$,	$*4\frac{3}{4}$	$3\frac{1}{2}$	$1\frac{1}{2}$	$\frac{1}{2}$	$1\frac{3}{8}$	$5\frac{3}{4}$	$6\frac{5}{8}$	$5\frac{1}{4}$	$\frac{1}{2}$	$3\frac{5}{16}$	4.6	PN 25
	5,	$*5\frac{1}{4}$	4	$1\frac{5}{8}$	$\frac{1}{2}$	$1\frac{1}{2}$	$6\frac{1}{4}$	$7\frac{3}{16}$	$5\frac{3}{4}$	$\frac{1}{2}$	$3\frac{13}{16}$	6.2	PN 26
$5\frac{1}{2}$,	$*5\frac{3}{4}$,	6	$4\frac{1}{2}$	$1\frac{3}{4}$	$\frac{1}{2}$	$1\frac{5}{8}$	7	$8\frac{1}{8}$	$6\frac{1}{2}$	$\frac{5}{8}$	$4\frac{5}{16}$	7.8	PN 27
	$*6\frac{1}{4}$,	$*6\frac{1}{2}$	5	$1\frac{7}{8}$	$\frac{3}{4}$	$1\frac{3}{4}$	$7\frac{5}{8}$	$8\frac{7}{8}$	7	$\frac{5}{8}$	$4\frac{13}{16}$	9.9	PN 28
	$*6\frac{3}{4}$,	7	$5\frac{1}{2}$	2	$\frac{3}{4}$	$1\frac{7}{8}$	$8\frac{1}{8}$	$9\frac{3}{8}$	$7\frac{1}{2}$	$\frac{3}{4}$	$5\frac{5}{16}$	11.8	PN 29
	$*7\frac{1}{4}$,	$*7\frac{1}{2}$	$5\frac{1}{2}$	2	$\frac{3}{4}$	$1\frac{7}{8}$	$8\frac{5}{8}$	10	8	$\frac{3}{4}$	$5\frac{5}{16}$	14.3	PN 30
$*7\frac{3}{4}$,	8,	$*8\frac{1}{4}$	6	$2\frac{1}{4}$	$\frac{3}{4}$	$2\frac{1}{8}$	$9\frac{3}{8}$	$10\frac{7}{8}$	$8\frac{3}{4}$	$\frac{3}{4}$	$5\frac{13}{16}$	18.6	PN 31
$*8\frac{1}{2}$,		9	6	$2\frac{1}{4}$	$\frac{3}{4}$	$2\frac{1}{8}$	$10\frac{1}{4}$	$11\frac{7}{8}$	$9\frac{5}{8}$	$\frac{3}{4}$	$5\frac{13}{16}$	23.8	PN 32
$*9\frac{1}{2}$,		10	6	$2\frac{3}{8}$	$\frac{3}{4}$	$2\frac{1}{4}$	$11\frac{1}{4}$	13	$10\frac{5}{8}$	$\frac{3}{4}$	$5\frac{13}{16}$	31.1	PN 33

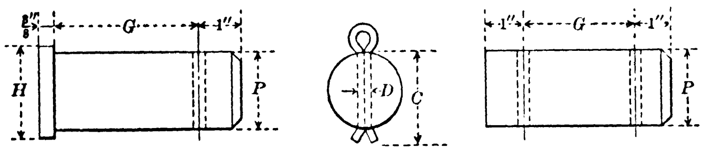
Pins marked * are special.

TABLE 96.

Cotter Pins.

AMERICAN BRIDGE COMPANY STANDARD.

All Dimensions in Inches.



Horizontal or Vertical Pin Finished.

Pin. P	Head. H	G	Cotter. C	Cotter. D
$1\frac{1}{4}$	$1\frac{1}{2}$	Net Grip + $\frac{1}{2}''$	2	$\frac{1}{4}$
$1\frac{1}{2}$	$1\frac{3}{4}$		$2\frac{1}{2}$	$\frac{1}{4}$
$1\frac{3}{4}$	2		$2\frac{3}{4}$	$\frac{1}{4}$
2	$2\frac{3}{8}$		3	$\frac{3}{8}$
$2\frac{1}{4}$	$2\frac{5}{8}$		$3\frac{1}{4}$	$\frac{3}{8}$
$2\frac{1}{2}$	$2\frac{7}{8}$		$3\frac{3}{4}$	$\frac{3}{8}$
$2\frac{3}{4}$	$3\frac{1}{8}$		4	$\frac{3}{8}$
3	$3\frac{1}{2}$		5	$\frac{1}{2}$
$3\frac{1}{4}$	$3\frac{3}{4}$		5	$\frac{1}{2}$
$3\frac{1}{2}$	4		6	$\frac{1}{2}$
$3\frac{3}{4}$	$4\frac{1}{4}$		6	$\frac{1}{2}$

Horizontal Pin Rough or Finished.

Pin. P	G	Cotter. C	Cotter. D
$1\frac{1}{4}$	Net Grip + $\frac{3}{4}''$	2	$\frac{1}{4}$
$1\frac{1}{2}$		$2\frac{1}{2}$	$\frac{1}{4}$
$1\frac{3}{4}$		$2\frac{3}{4}$	$\frac{1}{4}$
2		3	$\frac{3}{8}$
$2\frac{1}{4}$		$3\frac{1}{4}$	$\frac{3}{8}$
$2\frac{1}{2}$		$3\frac{3}{4}$	$\frac{3}{8}$
$2\frac{3}{4}$		4	$\frac{3}{8}$
3		5	$\frac{1}{2}$
$3\frac{1}{4}$		5	$\frac{1}{2}$
$3\frac{1}{2}$		6	$\frac{1}{2}$
$3\frac{3}{4}$		6	$\frac{1}{2}$

TABLE 97

Bearing Values of Pins.

Pin.		Bearing Value of Plate 1″ Thick for Unit Stress per Square Inch of					Diam. of Pin in In.
Diam. in In.	Area.	12 000	15 000	20 000	22 000	24 000	
1	.785	12 000	15 000	20 000	22 000	24 000	1
1¼	1.227	15 000	18 800	25 000	27 500	30 000	1¼
1½	1.767	18 000	22 500	30 000	33 000	36 000	1½
1¾	2.405	21 000	26 300	35 000	38 500	42 000	1¾
2	3.142	24 000	30 000	40 000	44 000	48 000	2
2¼	3.976	27 000	33 800	45 000	49 500	54 000	2¼
2½	4.909	30 000	37 500	50 000	55 000	60 000	2½
2¾	5.940	33 000	41 300	55 000	60 500	66 000	2¾
3	7.069	36 000	45 000	60 000	66 000	72 000	3
3¼	8.296	39 000	48 800	65 000	71 500	78 000	3¼
3½	9.621	42 000	52 500	70 000	77 000	84 000	3½
3¾	11.045	45 000	56 300	75 000	82 500	90 000	3¾
4	12.566	48 000	60 000	80 000	88 000	96 000	4
4¼	14.186	51 000	63 800	85 000	93 500	102 000	4¼
4½	15.904	54 000	67 500	90 000	99 000	108 000	4½
4¾	17.721	57 000	71 300	95 000	104 500	114 000	4¾
5	19.635	60 000	75 000	100 000	110 000	120 000	5
5¼	21.648	63 000	78 800	105 000	115 500	126 000	5¼
5½	23.758	66 000	82 500	110 000	121 000	132 000	5½
5¾	25.967	69 000	86 300	115 000	126 500	138 000	5¾
6	28.274	72 000	90 000	120 000	132 000	144 000	6
6¼	30.680	75 000	93 800	125 000	137 500	150 000	6¼
6½	33.183	78 000	97 500	130 000	143 000	156 000	6½
6¾	35.785	81 000	101 300	135 000	148 500	162 000	6¾
7	38.485	84 000	105 000	140 000	154 000	168 000	7
7¼	41.282	87 000	108 800	145 000	159 500	174 000	7¼
7½	44.179	90 000	112 500	150 000	165 000	180 000	7½
7¾	47.173	93 000	116 300	155 000	170 500	186 000	7¾
8	50.265	96 000	120 000	160 000	176 000	192 000	8
8¼	53.456	99 000	123 800	165 000	181 500	198 000	8¼
8½	56.745	102 000	127 500	170 000	187 000	204 000	8½
8¾	60.132	105 000	131 300	175 000	192 500	210 000	8¾
9	63.617	108 000	135 000	180 000	198 000	216 000	9
9¼	67.201	111 000	138 800	185 000	203 500	222 000	9¼
9½	70.882	114 000	142 500	190 000	209 000	228 000	9½
9¾	74.662	117 000	146 300	195 000	214 500	234 000	9¾
10	78.540	120 000	150 000	200 000	220 000	240 000	10
10¼	82.516	123 000	153 800	205 000	225 500	246 000	10¼
10½	86.590	126 000	157 500	210 000	231 000	252 000	10½
10¾	90.763	129 000	161 300	215 000	236 500	258 000	10¾
11	95.033	132 000	165 000	220 000	242 000	264 000	11
11¼	99.402	135 000	168 800	225 000	247 500	270 000	11¼
11½	103.869	138 000	172 500	230 000	253 000	276 000	11½
11¾	108.434	141 000	176 300	235 000	258 500	282 000	11¾
12	113.097	144 000	180 000	240 000	264 000	288 000	12

TABLE 98

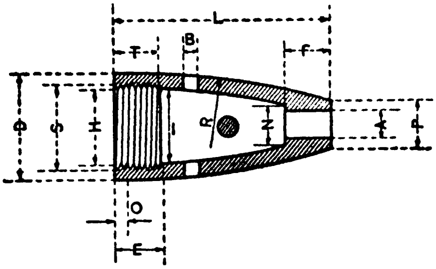
Bending Moments on Pins.

Pin.		Max. Moments in Inch-Pounds for Fiber Stress per Square Inch of							Diam. of Pin in In.
Diam. in In.	Area.	15 000	18 000	20 000	22 000	22 500	24 000	25 000	
1	.785	1 470	1 770	1 960	2 160	2 210	2 360	2 450	1
1¼	1.227	2 880	3 450	3 830	4 220	4 310	4 600	4 790	1¼
1½	1.767	4 970	5 960	6 630	7 290	7 460	7 950	8 280	1½
1¾	2.405	7 890	9 470	10 500	11 580	11 800	12 630	13 200	1¾
2	3.142	11 800	14 100	15 700	17 280	17 700	18 800	19 600	2
2¼	3.976	16 800	20 100	22 400	24 600	25 200	26 800	28 000	2¼
2½	4.909	23 000	27 600	30 700	33 700	34 500	36 800	38 300	2½
2¾	5.940	30 600	36 800	40 800	44 900	45 900	49 000	51 000	2¾
3	7.069	39 800	47 700	53 000	58 300	59 600	63 600	66 300	3
3¼	8.296	50 600	60 700	67 400	74 100	75 800	80 900	84 300	3¼
3½	9.621	63 100	75 800	84 200	92 600	94 700	101 000	105 200	3½
3¾	11.045	77 700	93 200	103 500	113 900	116 500	124 300	129 400	3¾
4	12.566	94 200	113 100	125 700	138 200	141 400	150 800	157 100	4
4¼	14.186	113 000	135 700	150 700	165 800	169 600	180 900	188 400	4¼
4½	15.904	134 200	161 000	178 900	196 800	201 300	214 700	223 700	4½
4¾	17.721	157 800	189 400	210 400	231 500	236 700	252 500	263 000	4¾
5	19.635	184 100	220 900	245 400	270 000	276 100	294 500	306 800	5
5¼	21.648	213 100	255 700	284 100	312 500	319 600	340 900	355 200	5¼
5½	23.758	245 000	294 000	326 700	359 300	367 500	392 000	408 300	5½
5¾	25.967	280 000	336 000	373 300	410 600	419 900	447 900	466 600	5¾
6	28.274	318 100	381 700	424 100	466 500	477 100	508 900	530 100	6
6¼	30.680	359 500	431 400	479 400	527 300	539 300	575 200	599 200	6¼
6½	33.183	404 400	485 300	539 200	593 100	606 600	647 100	674 000	6½
6¾	35.785	452 900	543 500	603 900	664 300	679 400	724 600	754 800	6¾
7	38.485	505 100	606 100	673 500	740 800	757 700	808 200	841 800	7
7¼	41.282	561 200	673 400	748 200	823 100	841 800	897 900	935 300	7¼
7½	44.179	621 300	745 500	828 400	911 200	931 900	994 000	1 035 400	7½
7¾	47.173	685 500	822 600	914 000	1 005 400	1 028 200	1 096 800	1 142 500	7¾
8	50.265	754 000	904 800	1 005 300	1 105 800	1 131 000	1 206 400	1 256 600	8
8¼	53.456	826 900	992 300	1 102 500	1 212 800	1 240 400	1 323 000	1 378 200	8¼
8½	56.745	904 400	1 085 300	1 205 800	1 326 400	1 356 600	1 447 000	1 507 300	8½
8¾	60.132	986 500	1 183 900	1 315 400	1 446 900	1 479 800	1 578 500	1 644 200	8¾
9	63.617	1 073 500	1 288 200	1 431 400	1 574 500	1 610 300	1 717 700	1 789 200	9
9¼	67.201	1 165 500	1 398 600	1 554 000	1 709 400	1 748 300	1 864 800	1 942 500	9¼
9½	70.882	1 262 600	1 515 100	1 683 500	1 851 800	1 893 900	2 020 100	2 104 300	9½
9¾	74.662	1 364 900	1 637 900	1 819 900	2 001 900	2 047 400	2 183 900	2 274 900	9¾
10	78.540	1 472 600	1 767 100	1 963 500	2 159 800	2 208 900	2 356 200	2 454 400	10
10¼	82.516	1 585 900	1 903 000	2 114 500	2 325 900	2 378 800	2 537 400	2 643 100	10¼
10½	86.590	1 704 700	2 045 700	2 273 000	2 500 300	2 557 100	2 727 600	2 841 200	10½
10¾	90.763	1 829 400	2 195 300	2 439 200	2 683 200	2 744 100	2 927 100	3 049 100	10¾
11	95.033	1 960 100	2 352 100	2 613 400	2 874 800	2 940 100	3 136 100	3 266 800	11
11¼	99.402	2 096 800	2 516 100	2 795 700	3 075 200	3 145 100	3 354 800	3 494 600	11¼
11½	103.869	2 239 700	2 687 600	2 986 200	3 284 900	3 359 500	3 583 500	3 732 800	11½
11¾	108.434	2 388 900	2 866 700	3 185 200	3 503 800	3 583 400	3 822 300	3 981 600	11¾
12	113.097	2 544 700	3 053 600	3 392 900	3 732 200	3 817 000	4 071 500	4 241 200	12

TABLE 99.

Long Pilot Nuts.

American Bridge Company's Standards.

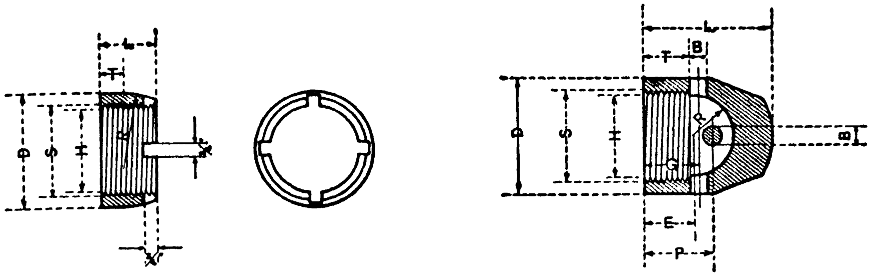


Pilot Nuts are made from Special Hard Steel and finished all over.

Screw, 6 Threads per Inch.

Diam. of Nut. D	Diam. of Screw. S	Diam. of Rough Hole. H	Length of Thread. T	Length over All. L	Straight Portion Inside. E	Diam. Inside. I	Straight Portion Outside. O	P	Radius. R	F	Clear. for Nut. N	Diam. of Hole. A	Diam. of Holes. B	Weight in Pounds.	Diam. of Nut. D
2"	1½"	1 5/16"	2"	5"	2"	.....	¾"	1"	18½"	1"	.....	½"	¾"	1.5	2"
2¼	“	“	“	“	“	.....	¼	“	“	“	.....	“	“	2.	2¼
2½	2	1 13/16	“	6	“	.....	½	“	20½	1⅛	.....	⅝	“	3.	2½
2¾	“	“	“	“	“	.....	1/16	“	“	“	.....	“	“	4.	2¾
3	2½	2 5/16	“	7	“	.....	1 1/16	1¼	“	1¼	.....	¾	“	5.	3
3¼	“	“	“	“	“	.....	11/16	“	“	“	.....	“	“	7.	3¼
3½	“	“	“	“	“	.....	5/16	“	“	“	....	“	“	9.	3½
3¾	3	2 13/16	2⅜	8	2½	.....	1 5/16	1½	“	2	.....	⅞	“	11.	3¾
4	“	“	“	“	“	.....	15/16	“	“	“	.....	“	“	12.	4
4¼	3½	3 5/16	“	9	“	.....	⅞	1¾	27	“	1½"	1	“	14.	4¼
4½	“	“	“	“	“	.....	½	“	“	“	“	“	“	16.	4½
4¾	“	“	“	“	“	.....	⅛	“	“	“	“	“	“	19.	4¾
5	4	3 13/16	“	10	“	.....	1⅛	2	“	“	1⅝	“	1	24.	5
5¼	“	“	“	“	“	.....	¾	“	“	“	“	“	“	30.	5¼
5½	4½	4 5/16	“	11½	“	.....	1⅛	2¾	40	2½	2 7/16	1½	“	33.	5½
5¾	“	“	“	“	“	.....	⅝	“	“	“	“	“	“	40.	5¾
6	“	“	“	“	2½	.....	3/16	“	“	“	“	“	“	45.	6
6¼	5	4 13/16	2⅞	13	3	.....	1 5/16	“	“	“	“	“	“	49.	6¼
6½	“	“	“	“	“	.....	⅞	“	“	“	“	“	“	58.	6½
6¾	5½	5 5/16	“	14½	“	.....	1½	“	43	“	“	“	“	64.	6¾
7	“	“	“	“	“	.....	1⅛	“	“	“	“	“	“	70.	7
7¼	“	“	“	“	“	.....	¾	“	“	“	“	“	“	77.	7¼
7½	“	“	“	“	“	.....	⅜	“	“	“	“	“	“	85.	7½
7¾	6	5 13/16	“	16	“	.....	1 9/16	“	“	“	“	“	“	95.	7¾
8	“	“	“	“	“	.....	1 3/16	“	“	“	“	“	“	102.	8
8¼	“	“	“	“	“	.....	⅞	“	“	“	“	“	“	110.	8¼
8½	“	“	2½	17½	......	6½"	13/16	3	52	2¾	2½	“	“	92.	8½
8¾	“	“	“	“	......	“	7/16	“	“	“	“	“	“	99.	8¾
9	“	“	“	“	......	“	⅛	“	“	“	“	“	“	107.	9
9¼	“	“	3	19½	......	7¼	1¼	“	“	“	“	“	1½	119.	9¼
9½	“	“	“	“	......	“	1⅜	“	“	“	“	“	“	130.	9½
9¾	“	“	“	“	......	“	1 1/16	“	“	“	“	“	“	142.	9¾
10	“	“	“	“	......	“	¾	“	“	“	“	“	“	156.	10
10¼	“	“	“	21½	......	8	1½	“	57	“	“	“	“	160.	10¼
10½	“	“	“	“	......	“	1 3/16	“	“	“	“	“	“	172.	10½
10¾	“	“	“	“	......	“	11/16	“	“	“	“	“	“	186.	10¾
11	“	“	“	“	......	“	½	“	“	“	“	“	“	203.	11

TABLE 100
Short Pilot Nuts and Driving Nuts.
American Bridge Company's Standards.



Dimensions in Inches.

Diam. of Nut. D	Diam. of Screw. S	Diam. of Rough Hole. H	Length over All. L	Straight Part. T	Radius. R	Weight in Pounds.
2″	1½″	1 5/16″	2″	1¼″	4¼″	5.
2¼	“	“	“	“	“	1.
2½	2	1 13/16	“	1	“	1.
2¾	“	“	“	“	“	1.5
3	2½	2 5/16	“	“	“	1.3
3¼	“	“	“	“	“	2.
3½	“	“	“	“	“	3.
3¾	3	2 13/16	2½	“	“	3.
4	“	“	“	“	“	4.
4¼	3½	3 5/16	“	“	“	3.
4½	“	“	“	“	“	4.
4¾	“	“	“	“	“	5.
5	4	3 13/16	“	“	“	5.
5¼	“	“	“	“	“	6.
5½	4½	4 5/16	“	“	“	5.
5¾	“	“	“	“	“	6.
6	“	“	“	“	“	8.
6¼	5	4 13/16	3	1¼	“	9.
6½	“	“	“	“	“	11.
6¾	5½	5 5/16	“	“	“	10.
7	“	“	“	“	“	12.
7¼	“	“	“	“	“	14.
7½	“	“	“	“	“	16.
7¾	6	5 13/16	“	½	6½	14.
8	“	“	“	“	“	16.
8¼	“	“	“	“	“	19.
8½	“	“	“	“	“	21.
8¾	“	“	“	“	“	24.
9	“	“	“	“	“	28.
9¼	“	“	3½	“	7	33.
9½	“	“	“	“	“	36.
9¾	“	“	“	“	“	40.
10	“	“	“	“	“	45.
10¼	“	“	“	“	“	48.
10½	“	“	“	“	“	51.
10¾	“	“	“	“	“	55.
11	“	“	“	“	“	59.

Dimensions in Inches.

Diam. of Nut. D	Diam. of Screw. S	Diam. of Rough Hole. H	Length of Thread. T	Length over All. L	Straight Part Inside. E	Inside Radius. R	G	P	Diam. of Holes. B	Weight in Pounds.
2⅜″	1½″	1 5/16″	2″	4″	2¼″	21/32″	2″	2 9/16″	¾″	4
2⅞	2	1 13/16	“	4⅜	“	29/32	2 1/16	2¾	“	5
3⅝	2½	2 5/16	“	5	2⅝	1 5/32	2⅜	3 3/16	“	8
4⅛	3	2 13/16	2⅜	5¼	“	1 13/32	2½	3⅜	“	11
4⅞	3½	3 5/16	“	5⅞	“	1 21/32	“	3½	“	17
5⅜	4	3 11/16	“	6⅜	“	1 29/32	2⅝	3¾	1	22
6¼	4½	4 5/16	“	6¾	“	2 5/32	2¾	3⅞	“	27
6¾	5	4 13/16	2⅞	7⅛	3	2 13/32	3⅜	4⅝	“	37
7¾	5½	5 5/16	“	7⅝	“	2 21/32	“	“	“	56
8½	6	5 13/16	“	7⅞	“	2 29/32	“	“	“	67
9¼	“	“	“	“	“	“	“	4⅞	1½	86
10¼	“	“	3	8⅜	3½	“	4	5½	“	120
11¼	“	“	“	“	“	“	“	“	“	150

Pilot Nuts and Driving Nuts are made from special hard steel. Pilot nuts are finished all over.

Screws 6 threads per inch.

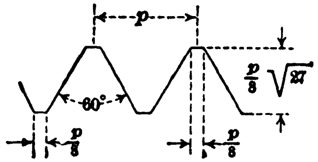
When short pilot nuts are needed on bottom chord pins, long pilot nuts are to be sent for all other pins, in addition.

TABLE 101.

Screw Threads.

AMERICAN BRIDGE COMPANY STANDARD.

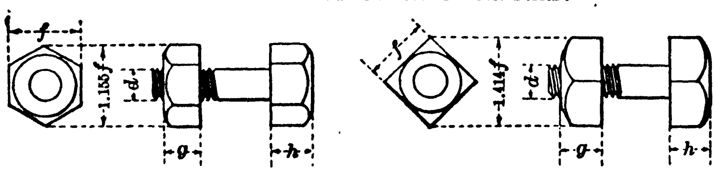
Bolts, Rods, Eye Bars, Turnbuckles, Sleeve Nuts, and Clevises.



Diameter. Total d, In.	Diameter. Net, c, In.	Area. Total Dia., d, Sq. In.	Area. Net Dia., c, Sq. In.	Number of Threads per Inch.	Diameter. Total, d, In.	Diameter. Net, c, In.	Area. Total Dia., d, Sq. In.	Area. Net Dia., c, Sq. In.	Number of Threads per Inch.
1/4	.185	.049	.027	20	2 1/2	2.175	4.909	3.716	4
3/8	.294	.110	.068	16	2 5/8	2.300	5.412	4.156	4
1/2	.400	.196	.126	13	2 3/4	2.425	5.940	4.619	4
5/8	.507	.307	.202	11	2 7/8	2.550	6.492	5.108	4
3/4	.620	.442	.302	10					
7/8	.731	.601	.419	9	3	2.629	7.069	5.428	3 1/2
					3 1/4	2.879	8.296	6.509	3 1/2
1	.838	.785	.551	8	3 1/2	3.100	9.621	7.549	3 1/4
1 1/8	.939	.994	.693	7	3 3/4	3.317	11.045	8.641	3
1 1/4	1.064	1.227	.890	7					
1 3/8	1.158	1.485	1.054	6	4	3.567	12.566	9.993	3
1 1/2	1.283	1.767	1.294	6	4 1/4	3.798	14.186	11.330	2 7/8
1 5/8	1.389	2.074	1.515	5 1/2	4 1/2	4.028	15.904	12.741	2 3/4
1 3/4	1.490	2.405	1.744	5	4 3/4	4.255	17.721	14.221	2 5/8
1 7/8	1.615	2.761	2.049	5					
					5	4.480	19.635	15.766	2 1/2
2	1.711	3.142	2.300	4 1/2	5 1/4	4.730	21.648	17.574	2 1/2
2 1/8	1.836	3.547	2.649	4 1/2	5 1/2	4.953	23.758	19.268	2 3/8
2 1/4	1.961	3.976	3.021	4 1/2	5 3/4	5.203	25.967	21.262	2 3/8
2 3/8	2.086	4.430	3.419	4 1/2	6	5.423	28.274	23.095	2 1/4

Bolt Heads and Nuts.

AMERICAN BRIDGE COMPANY STANDARD.



Rough Nut.		Finished Nut.		Rough Head.		Finished Head.	
f	g	f	g	f	h	f	h
1.5d + 1/8″	d	1.5d + 1/16″	d − 1/16″	1.5d + 1/8″	0.5f	1.5d + 1/16″	0.5f − 1/16″

For Screw Threads, Bolt Heads and Nuts, the American Bridge Company has adopted the Franklin Institute Standard, commonly known as United States Standard.

TABLE 102.

Bolt Heads and Nuts, Dimensions in Inches.

AMERICAN BRIDGE COMPANY STANDARD.

Diameter of Bolt, Inches.	HEAD. Hexagonal. Diameter. Long.	HEAD. Hexagonal. Diameter. Short.	HEAD. Hex. or Square. Height.	HEAD. Square. Diameter. Long.	HEAD. Square. Diameter. Short.	Diameter of Bolt, Inches.	NUT. Hexagonal. Diameter. Long.	NUT. Hexagonal. Diameter. Short.	NUT. Hex. or Square. Height.	NUT. Square. Diameter. Long.	NUT. Square. Diameter. Short.
1/4	5/8	1/2	1/4	11/16	1/2	1/4	5/8	1/2	1/4	11/16	1/2
3/8	13/16	11/16	3/8	1	11/16	3/8	13/16	11/16	3/8	1	11/16
1/2	1	7/8	7/16	1 1/4	7/8	1/2	1	7/8	1/2	1 1/4	7/8
5/8	1 1/4	1 1/16	9/16	1 1/2	1 1/16	5/8	1 1/4	1 1/16	5/8	1 1/2	1 1/16
3/4	1 7/16	1 1/4	5/8	1 13/16	1 1/4	3/4	1 7/16	1 1/4	3/4	1 13/16	1 1/4
7/8	1 11/16	1 7/16	3/4	2 1/16	1 7/16	7/8	1 11/16	1 7/16	7/8	2 1/16	1 7/16
1	1 7/8	1 5/8	13/16	2 5/16	1 5/8	1	1 7/8	1 5/8	1	2 5/16	1 5/8
1 1/8	2 1/8	1 13/16	15/16	2 9/16	1 13/16	1 1/8	2 1/8	1 13/16	1 1/8	2 9/16	1 13/16
1 1/4	2 5/16	2	1	2 13/16	2	1 1/4	2 5/16	2	1 1/4	2 13/16	2
1 3/8	2 9/16	2 3/16	1 1/8	3 1/8	2 3/16	1 3/8	2 9/16	2 3/16	1 3/8	3 1/8	2 3/16
1 1/2	2 3/4	2 3/8	1 3/16	3 3/8	2 3/8	1 1/2	2 3/4	2 3/8	1 1/2	3 3/8	2 3/8
1 5/8	3	2 9/16	1 5/16	3 5/8	2 9/16	1 5/8	3	2 9/16	1 5/8	3 5/8	2 9/16
1 3/4	3 3/16	2 3/4	1 3/8	3 7/8	2 3/4	1 3/4	3 3/16	2 3/4	1 3/4	3 7/8	2 3/4
1 7/8	3 7/16	2 15/16	1 1/2	4 3/16	2 15/16	1 7/8	3 7/16	2 15/16	1 7/8	4 3/16	2 15/16
2	3 5/8	3 1/8	1 9/16	4 7/16	3 1/8	2	3 5/8	3 1/8	2	4 7/16	3 1/8
2 1/4	4 1/16	3 1/2	1 3/4	4 15/16	3 1/2	2 1/4	4 1/16	3 1/2	2 1/4	4 15/16	3 1/2
2 1/2	4 1/2	3 7/8	1 15/16	5 1/2	3 7/8	2 1/2	4 1/2	3 7/8	2 1/2	5 1/2	3 7/8
2 3/4	4 15/16	4 1/4	2 1/8	6	4 1/4	2 3/4	4 15/16	4 1/4	2 3/4	6	4 1/4
3	5 3/8	4 5/8	2 5/16	6 9/16	4 5/8	3	5 3/8	4 5/8	3	6 9/16	4 5/8
3 1/4	5 13/16	5	2 1/2	7 1/16	5	3 1/4	5 13/16	5	3 1/4	7 1/16	5
3 1/2	6 1/4	5 3/8	2 11/16	7 5/8	5 3/8	3 1/2	6 1/4	5 3/8	3 1/2	7 5/8	5 3/8

Bolt Threads, Length in Inches.

Length, Inches.	Diameter, Inches. 1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4
1 to 1 1/2	3/4	3/4	1	1 1/4	....	....	....	....	....
1 5/8 to 2	3/4	3/4	1	1 1/4	1 1/2	1 1/2	....	....	....
2 1/8 to 2 1/2	3/4	3/4	1	1 1/4	1 1/2	1 3/4	1 3/4	....	....
2 5/8 to 3	7/8	7/8	1	1 1/4	1 1/2	1 3/4	1 3/4	2 1/4	....
3 1/8 to 4	7/8	7/8	1 1/4	1 1/4	1 1/2	1 3/4	1 3/4	2 1/4	2 1/2
4 1/8 to 8	1	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	2 3/4
8 1/8 to 12	1	1	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3
12 1/8 to 20	1	1	1 1/2	2	2	2 1/4	2 1/2	3	3

Bolts not listed are threaded about 3 times the diameter; in no case are standard bolts threaded closer to the head than 1/2 inch.

TABLE 103.

Bolts with Hexagon Heads and Nuts.

AMERICAN BRIDGE COMPANY STANDARD.

Weight in Pounds per 100 Bolts.

Length Under Head, Inches.	Diameter of Bolt, Inches.					Length Under Head, Inches.	Diameter of Bolt, Inches.				
	½	⅝	¾	⅞	1		½	⅝	¾	⅞	1
1	19	33	52	......	......	8	58	92	137	194	264
1¼	20	34	54	......	......	8½	60	96	143	202	274
1½	22	36	57	......	......	9	63	100	149	210	285
1¾	23	38	60	......	......	9½	66	105	156	219	296
2	24	40	63	93	132	10	68	109	162	227	307
2¼	26	43	66	97	137	10½	71	114	168	236	318
2½	27	45	69	101	143	11	74	118	174	244	329
2¾	29	47	72	105	148	11½	77	122	181	253	341
3	30	49	75	109	154	12	80	127	187	261	352
3¼	31	51	78	114	160	12½	82	131	193	270	363
3½	33	54	82	118	165	13	85	135	199	278	374
3¾	34	56	85	122	171	13½	88	139	206	287	385
4	35	58	88	126	176	14	91	144	212	295	396
4¼	37	60	90	130	180	14½	93	148	218	304	407
4½	38	62	94	134	186	15	96	152	225	312	418
4¾	39	64	97	138	191	15½	99	157	231	321	430
5	41	66	100	143	197	16	102	161	237	329	441
5¼	42	68	103	147	202	16½	105	165	243	338	452
5½	44	71	106	151	208	17	107	170	250	346	463
5¾	45	73	109	156	213	17½	110	174	256	355	474
6	46	75	112	160	219	18	113	177	262	364	485
6¼	48	77	115	164	225	18½	116	183	268	372	496
6½	49	79	119	168	230	19	119	187	275	381	507
6¾	51	81	122	173	236	19½	121	191	281	389	519
7	52	84	125	177	241	20	124	196	287	398	530
7¼	53	86	128	181	247	....	......	......	......	......	......
7½	55	88	131	185	252	....	......	......	......	......	......
7¾	56	90	134	190	258	....	......	......	......	......	......
Per Inch Additional	5.6	8.7	12.5	17.0	22.3	Per Inch Additional	5.6	8.7	12.5	17.0	22.3

Hexagon Nuts and Bolt Heads.

Weights in Pounds for One Head and One Nut.

Diameter of Bolt, Inches.	1¼	1½	1¾	2	2½	3
Hexagon Head and Nut..........	1.73	2.95	4.61	6.79	13.0	22.0
Weight of Shank per Inch........	.3479	.5007	.6815	.8900	1.391	2.003

TABLE 104.

Bolts with Square Heads and Nuts.

American Bridge Company Standard.

Weight in Pounds per 100 Bolts.

Length Under Head, Inches.	Diameter of Bolt, Inches.								
	1/4	5/16	3/8	7/16	1/2	5/8	3/4	7/8	1
1	4	7	11	15	22	37	56	......	......
1 1/4	4	7	11	16	23	39	59	......	......
1 1/2	5	8	12	17	24	41	62	......	......
1 3/4	5	8	13	18	26	43	64	......	......
2	5	9	14	19	27	45	67	101	144
2 1/4	6	9	15	20	28	47	71	104	150
2 1/2	6	10	15	21	30	49	74	109	155
2 3/4	6	10	16	22	31	51	77	113	161
3	7	11	17	24	33	54	80	117	167
3 1/2	7	12	18	25	35	58	86	126	178
4	8	13	20	28	38	62	92	134	189
4 1/2	9	14	21	30	41	66	98	142	198
5	10	15	23	32	43	71	104	151	209
5 1/2	10	16	25	34	46	75	111	159	220
6	11	17	26	36	49	79	117	168	232
6 1/2	......	......	28	38	52	84	123	176	243
7	......	......	29	40	55	88	129	185	254
7 1/2	......	......	31	42	57	92	136	193	265
8	......	......	32	45	60	97	142	202	276
9	......	......	34	49	65	105	154	218	298
10	......	......	......	53	71	114	167	235	320
12	......	......	......	61	82	131	192	269	364
14	......	......	......	......	93	148	217	303	409
Per Inch Additional...	1.4	2.2	3.1	4.3	5.6	8.7	12.5	17.0	22.3

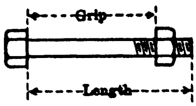
Square Nuts and Bolt Heads.

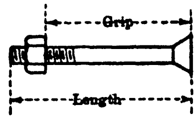
Weights in Pounds for One Head and One Nut.

Diameter of Bolt, Inches.	1 1/4	1 1/2	1 3/4	2	2 1/2	3
Square Head and Nut...........	2.05	3.51	5.48	8.08	15.5	26.2
Weight of Shank per Inch........	.3477	.5007	.6815	.8900	1.391	2.003

TABLE 105.

Lengths of Bolts and Tie Rods.



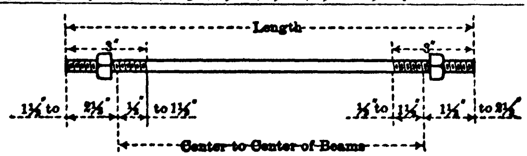


Grip.	Diameter.				
	½″	⅝″	¾″	⅞″	1″
½″	1¼″	1¼″	1½″	1½″	1¾″
⅝	1¼	1½	1½	1¾	1¾
¾	1½	1½	1¾	1¾	2
⅞	1½	1¾	1¾	2	2
1	1¾	1¾	2	2	2¼
⅛	1¾	2	2	2¼	2¼
¼	2	2	2¼	2¼	2½
⅜	2	2¼	2¼	2½	2½
½	2¼	2¼	2½	2½	2¾
⅝	2¼	2½	2½	2¾	2¾
¾	2½	2½	2¾	2¾	3
⅞	2½	2¾	2¾	3	3
2	2¾	2¾	3	3	3¼
⅛	2¾	3	3	3¼	3¼
¼	3	3	3¼	3¼	3½
⅜	3	3¼	3¼	3½	3½
½	3¼	3¼	3½	3½	3¾
⅝	3¼	3½	3½	3¾	3¾
¾	3½	3½	3¾	3¾	4
⅞	3½	3¾	3¾	4	4
3	4	4	4	4	4½
⅛	4	4	4	4½	4½
¼	4	4	4½	4½	4½
⅜	4	4½	4½	4½	4½
½	4½	4½	4½	4½	5
⅝	4½	4½	4½	5	5
¾	4½	4½	5	5	5
⅞	4½	5	5	5	5

Grip.	Diameter.				
	½″	⅝″	¾″	⅞″	1″
4″	5″	5″	5″	5″	5½″
⅛	5	5	5	5½	5½
¼	5	5	5½	5½	5½
⅜	5	5½	5½	5½	5½
½	5½	5½	5½	5½	6
⅝	5½	5½	5½	6	6
¾	5½	5½	6	6	6
⅞	5½	6	6	6	6
5	6	6	6	6	6½
⅛	6	6	6	6½	6½
¼	6	6	6½	6½	6½
⅜	6	6½	6½	6½	6½
½	6½	6½	6½	6½	7
⅝	6½	6½	6½	7	7
¾	6½	6½	7	7	7
⅞	6½	7	7	7	7
6	7	7	7	7	7½
⅛	7	7	7	7½	7½
¼	7	7	7½	7½	7½
⅜	7	7½	7½	7½	7½
½	7½	7½	7½	7½	8
⅝	7½	7½	7½	8	8
¾	7½	7½	8	8	8
⅞	7½	8	8	8	8
7	8	8	8	8	8½
⅛	8	8	8	8½	8½
¼	8	8	8½	8½	8½
⅜	8	8½	8½	8½	8½
½	8½	8½	8½	8½	9
⅝	8½	8½	8½	9	9
¾	8½	8½	9	9	9
⅞	8½	9	9	9	9

Grip.	Diameter.				
	½″	⅝″	¾″	⅞″	1″
8″	9″	9″	9″	9″	9½″
⅛	9	9	9	9½	9½
¼	9	9	9½	9½	9½
⅜	9	9½	9½	9½	9½
½	9½	9½	9½	9½	10
⅝	9½	9½	9½	10	10
¾	9½	9½	10	10	10
⅞	9½	10	10	10	10
9	10	10	10	10	10½
⅛	10	10	10	10½	10½
¼	10	10	10½	10½	10½
⅜	10	10½	10½	10½	10½
½	10½	10½	10½	10½	11
⅝	10½	10½	10½	11	11
¾	10½	10½	11	11	11
⅞	10½	11	11	11	11
10	11	11	11	11	11½
⅛	11	11	11	11½	11½
¼	11	11	11½	11½	11½
⅜	11	11½	11½	11½	11½
½	11½	11½	11½	11½	12
⅝	11½	11½	11½	12	12
¾	11½	11½	12	12	12
⅞	11½	12	12	12	12
11	12	12	12	12	....
⅛	12	12	12	.....	....
¼	12	12	.....	.....	...
⅜	12	.....	.....	.....	....

For Cut Threads use ⅝″, ¾″ and ⅞″ Rods



For Rolled Threads use 11/16″ instead of ¾″ Rods, 13/16″ instead of ⅞″ Rods

C to C Beams.	Lgth.	C to C Beams.	Lgth.	C to C Beams.	Lgth.	C to C Beams.	Lgth.	C to C Beams.	Lgth.	C to C Beams.	Lgth.
′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″
1-0	1-3	2-4, 5, 6	2-9	3-10, 11	4-3	5-1, 2, 3	5-6	6-7, 8, 9	7-0	8-0	8-3
1-1, 2, 3	1-6	2-7, 8, 9	3-0	4-0	4-3	5-4, 5, 6	5-9	6-10, 11	7-3	8-1, 2, 3	8-6
1-4, 5, 6	1-9	2-10, 11	3-3	4-1, 2, 3	4-6	5-7, 8, 9	6-0	7-0	7-3	8-4, 5, 6	8-9
1-7, 8, 9	2-0	3-0	3-3	4-4, 5, 6	4-9	5-10, 11	6-3	7-1, 2, 3	7-6	8-7, 8, 9	9-0
1-10, 11	2-3	3-1, 2, 3	3-6	4-7, 8, 9	5-0	6-0	6-3	7-4, 5, 6	7-9	8-10, 11	9-3
2-0	2-3	3-4, 5, 6	3-9	4-10, 11	5-3	6-1, 2, 3	6-6	7-7, 8, 9	8-0		
2-1, 2, 3	2-6	3-7, 8, 9	4-0	5-0	5-3	6-4, 5, 6	6-9	7-10, 11	8-3		

TABLE 106.

Structural Rivets.

AMERICAN BRIDGE COMPANY STANDARD.

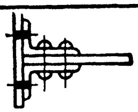
Weight in Pounds per 100 Rivets with Button Heads.

Length Under Head, Inches.	Diameter of Rivet, Inches. ⅜	½	⅝	¾	⅞	1	1⅛	1¼	Length Under Head, Inches.	Diameter of Rivet, Inches. ⅜	½	⅝	¾	⅞	1	1⅛	1¼
									5	18	33	53	78	109	146	190	252
									⅛	18	34	54	80	111	149	193	256
1¼	6	12	.....	.....	.....	.....	.....	.....	¼	19	34	55	82	113	152	197	260
⅜	7	13	.....	.....	.....	.....	.....	.....	⅜	19	35	56	83	115	155	200	265
½	7	13	23	35	50	68	91	130	½	20	36	57	85	118	157	204	269
⅝	7	14	24	36	52	71	95	134	⅝	20	36	58	86	120	160	207	273
¾	8	15	25	37	54	74	98	139	¾	20	37	60	88	122	163	211	278
⅞	8	15	26	39	56	77	102	143	⅞	21	38	61	89	124	166	214	282
2	9	16	27	41	58	80	105	148	6	21	38	62	91	126	169	218	287
⅛	9	17	28	43	60	82	109	152	⅛	22	39	63	93	128	171	222	291
¼	9	18	29	44	62	85	112	156	¼	22	40	64	94	130	174	225	295
⅜	10	18	30	46	64	88	116	161	⅜	22	40	65	96	132	177	229	300
½	10	19	31	47	67	91	119	165	½	23	41	66	97	135	180	232	304
⅝	11	20	32	49	69	93	123	169	⅝	23	42	67	99	137	182	236	308
¾	11	20	34	50	71	96	126	174	¾	24	43	68	100	139	185	239	313
⅞	11	21	35	52	73	99	130	178	⅞	24	43	69	102	141	188	243	317
3	12	22	36	54	75	102	133	182	7	24	44	70	104	143	191	246	321
⅛	12	22	37	55	77	105	137	187	⅛	25	45	71	105	145	194	250	326
¼	13	23	38	57	79	107	141	191	¼	25	45	73	107	147	196	253	330
⅜	13	24	39	58	81	110	144	195	⅜	26	46	74	108	149	199	257	334
½	13	24	40	60	84	113	148	200	½	26	47	75	110	152	202	260	339
⅝	14	25	41	61	86	116	151	204	⅝	26	47	76	111	154	205	264	343
¾	14	26	42	63	88	118	155	208	¾	27	48	77	113	156	207	267	347
⅞	15	27	43	64	90	121	158	213	⅞	27	49	78	114	158	210	271	352
4	15	27	44	66	92	124	162	217	8	27	50	79	116	160	213	274	356
⅛	15	28	45	68	94	127	165	221	⅛	28	50	80	118	162	216	278	360
¼	16	29	47	69	96	130	169	226	¼	28	51	81	119	164	219	281	365
⅜	16	29	48	71	98	132	172	230	⅜	29	52	82	121	166	221	285	369
½	16	30	49	72	101	135	176	234	½	29	52	83	122	169	224	288	373
⅝	17	31	50	74	103	138	179	239	⅝	29	53	84	124	171	227	292	378
¾	17	31	51	75	105	141	183	243	¾	30	54	86	125	173	230	295	382
⅞	18	32	52	77	107	143	186	247	⅞	30	54	87	127	175	232	299	386

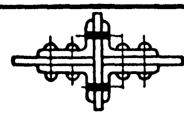
Button Heads.	Diameter of Rivets, Inches. ⅜	½	⅝	¾	⅞	1	1⅛	1¼
100 Heads as made on rivets, Pounds...	2.4	5.0	9.7	16.0	24.0	35.0	49.0	78.0
100 Heads as driven in work, Pounds...	1.9	4.0	7.5	12.5	18.5	27.0	37.5	51.0

TABLE 107.

Lengths of Field Rivets and Bolts for Beam Framing.



⅞" Rivets.



	Single. Bolt In.	Single. Riv. In.	24"	20"	18"	15"	12"	10"	9"	8"	7"	6"	5"	4"	3"	Double. Riv. In.	Double. Bolt In.	
BEAMS	1½	2⅛										12.25	9.75	7.5 8.5	5.5 6.5	2½	2	BEAMS
	1¾	2⅛						25	21 25	18 20.5	15 17.5	14.75	12.25	9.5	7.5	2⅝	2¼	
		2¼				42	31.5			23		17.25		10.5		2¾		
	2	2⅜	80	65	55 60	45 50 60	35 40	30 35	35 30	25	20		14.75			2¾		
		2½	85 90	70 75 80 85	65 75 80	55 65	45									3	2½	
	2¼	2⅝	95 100 115	90 95	70 85	70 75 80	50 55 60	40								3⅛		
		2¾		100	90	85	65									3⅜	2¾	
		2⅞				90										3½		
	2½	3⅛				95 100										3⅝	3	
CHANNELS	1½	2										8.00	6.50	5.25	4.00	2⅜	2	CHANNELS
		2⅛					20.5	15	13.25	11.25 13.75	9.75			6.25	5.00	2½		
	1¾	2⅛							15.00		12.25	10.50	9.00	7.25	6.00	2⅝	2¼	
		2¼				33 35	25	20	20	16.25 18.75	14.75		11.50			2⅝		
	2	2¼				40		25				13				2¾		
		2⅜					30 35		25	21.25	17.25 19.75	15.50				2¾		
		2½				45 50	40	30								3	2½	
	2¼	2⅝				55		35								3⅛	2¾	

	24"	20"	18"	15"	12"	10"	9"	8"	7"	6"	5"	4"	3"	Riv. Top & Bott.	Bolt. Top & Bott.	
Top Angle = ⅜".									all	all	all	all	all	2¼	1¾	BEAMS
						all	all	all						2⅜	2	
				42 to 55	31.5 35									2½		
			55 to 70		40 to 65									2⅝	2¼	
	80 to 100	65 to 75		60 to 75										2¾		
	115	80 to 100		80 to 100										3	2½	
Bottom Angle = ½".													all	2⅛	1¾	CHANNELS
								all	all	all	all	all		2¼		
					20.5 25 30	all	all							2⅜	2	
				all	35 40									2½		

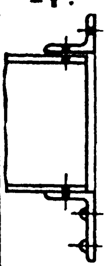


TABLE 108.

Structural Rivets.

AMERICAN BRIDGE COMPANY STANDARD.

Lengths of Field Rivets for Various Grips.

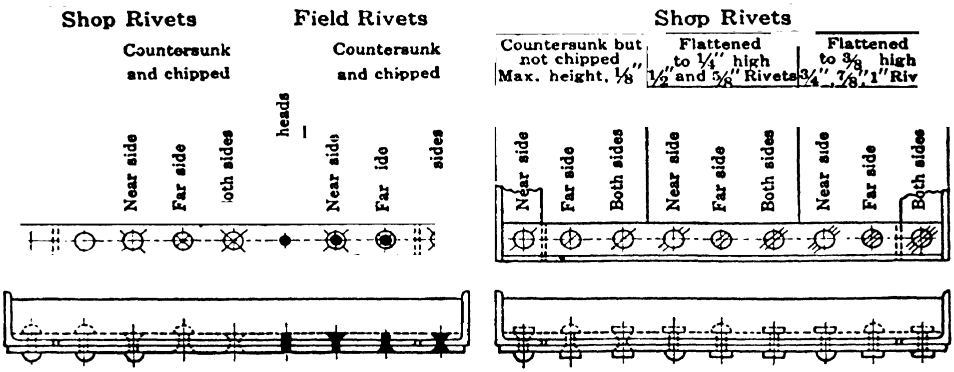
Dimensions in Inches.

Grip a.	Diameter.					Grip b.	Diameter.				
	½	⅝	¾	⅞	1		½	⅝	¾	⅞	1
½	1½	1¾	1⅞	2	2⅛	½	1⅛	1¼	1¼	1⅜	1⅜
⅝	1⅝	1⅞	2	2⅛	2¼	⅝	1¼	1⅜	1⅜	1½	1½
¾	1¾	2	2⅛	2¼	2⅜	¾	1⅜	1½	1½	1⅝	1⅝
⅞	1⅞	2⅛	2¼	2⅜	2½	⅞	1½	1⅝	1⅝	1¾	1¾
1	2	2¼	2⅜	2½	2⅝	1	1⅝	1¾	1¾	1⅞	1⅞
⅛	2⅛	2⅜	2½	2⅝	2¾	⅛	1¾	1⅞	1⅞	2	2
¼	2¼	2½	2⅝	2¾	2⅞	¼	1⅞	2	2	2⅛	2⅛
⅜	2⅜	2⅝	2¾	2⅞	3	⅜	2	2⅛	2⅛	2¼	2¼
½	2⅝	2⅞	3	3⅛	3¼	½	2⅛	2¼	2⅜	2⅜	2½
⅝	2¾	3	3⅛	3¼	3⅜	⅝	2¼	2⅜	2½	2½	2⅝
¾	3	3¼	3⅜	3½	3⅝	¾	2½	2⅝	2¾	2¾	2⅞
⅞	3⅛	3⅜	3½	3⅝	3¾	⅞	2⅝	2¾	2⅞	2⅞	3
2	3¼	3½	3⅝	3¾	3⅞	2	2¾	2⅞	3	3	3⅛
⅛	3⅜	3⅝	3¾	3⅞	4	⅛	2⅞	3	3⅛	3⅛	3¼
¼	3½	3¾	3⅞	4	4⅛	¼	3	3⅛	3¼	3¼	3⅜
⅜	3⅝	3⅞	4	4⅛	4¼	⅜	3⅛	3¼	3⅜	3⅜	3½
½	3¾	4	4⅛	4¼	4⅜	½	3¼	3⅜	3½	3½	3⅝
⅝	3⅞	4⅛	4¼	4⅜	4½	⅝	3⅜	3½	3⅝	3⅝	3¾
¾	4	4¼	4⅜	4½	4⅝	¾	3½	3⅝	3¾	3¾	3⅞
⅞	4⅛	4⅜	4½	4⅝	4¾	⅞	3⅝	3¾	3⅞	3⅞	4
3	4⅜	4⅝	4¾	4⅞	5	3	3⅞	4	4	4⅛	4¼
⅛	4½	4¾	4⅞	5	5⅛	⅛	4	4⅛	4⅛	4¼	4⅜
¼	4⅝	4⅞	5	5⅛	5¼	¼	4⅛	4¼	4¼	4⅜	4½
⅜	4¾	5	5⅛	5¼	5⅜	⅜	4¼	4⅜	4⅜	4½	4⅝
½	4⅞	5⅛	5¼	5⅜	5½	½	4⅜	4½	4½	4⅝	4¾
⅝	5	5¼	5⅜	5½	5⅝	⅝	4½	4⅝	4⅝	4¾	4⅞
¾	5⅛	5⅜	5½	5⅝	5¾	¾	4⅝	4¾	4¾	4⅞	5
⅞	5¼	5½	5⅝	5¾	5⅞	⅞	4¾	4⅞	4⅞	5	5⅛
4	5⅜	5⅝	5¾	5⅞	6	4	4⅞	5	5	5⅛	5¼
⅛	5⅝	5⅞	6	6⅛	6¼	⅛	5⅛	5¼	5¼	5⅜	5½
¼	5¾	6	6⅛	6¼	6⅜	¼	5¼	5⅜	5⅜	5½	5⅝
⅜	6	6¼	6⅜	6½	6⅝	⅜	5½	5⅝	5⅝	5⅝	5¾
½	6⅛	6⅜	6½	6⅝	6¾	½	5⅝	5¾	5¾	5¾	5⅞
⅝	6¼	6½	6⅝	6¾	6⅞	⅝	5¾	5⅞	5⅞	5⅞	6
¾	6⅜	6⅝	6¾	6⅞	7	¾	5⅞	6	6	6	6⅛
⅞	6½	6¾	6⅞	7	7⅛	⅞	6	6⅛	6⅛	6⅛	6¼
5	6⅝	6⅞	7	7⅛	7¼	5	6⅛	6¼	6¼	6¼	6⅜
⅛	......	......	7⅛	7¼	7⅜	⅛	......	......	6⅜	6⅜	6½
¼	......	......	7¼	7⅜	7½	¼	......	......	6½	6½	6⅝
⅜	......	......	7⅜	7½	7⅝	⅜	......	......	6⅝	6⅝	6¾
½	......	......	7⅝	7¾	7⅞	½	......	......	6⅞	6⅞	7
⅝	......	......	7¾	7⅞	8	⅝	......	......	7	7	7⅛
¾	......	......	7⅞	8	8⅛	¾	......	......	7⅛	7⅛	7¼
⅞	......	......	8	8⅛	8¼	⅞	......	......	7¼	7¼	7⅜

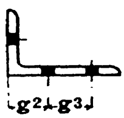
TABLE 109.

STANDARDS FOR RIVETS AND RIVETING.

CONVENTIONAL SIGNS FOR RIVETING



GAGES FOR ANGLES, INCHES



Leg	8	7	6	5	4	3½	3	2½	2	1¾	1½	1⅜	1¼	1	¾
g1	4½	4	3½	3	2½	2	1¾	1⅜	1⅛	1	⅞	⅞	¾	⅝	½
g2	3	2½	2½	2											
g3	3	3	2¼	1¾											
Max. rivet	1⅛	1	⅞	⅞	⅞	⅞	⅞	¾	⅝	½	⅜	⅜	⅜	¼	¼

For column details, 6″ leg (½ inch thick or less) against column shaft, $g^2 = 1\frac{3}{4}''$, $g^3 = 3''$.

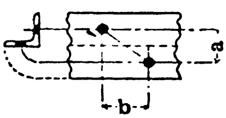
For diagonal angles, etc., gage in middle, where riveted leg equals or exceeds 3″ for ¾″ rivets 3½″ for ⅞″ rivets.

Use special gages to adapt work to multiple punch, or to secure desirable details.

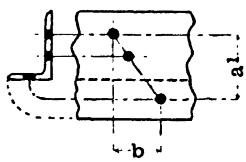
STAGGER OF RIVETS TO MAINTAIN NET SECTION

AMERICAN BRIDGE COMPANY STANDARD

1 Hole Out



2 Holes Out



Dimensions in Inches

	¾″ Rivet	⅞″ Rivet	a^1	¾″ Rivet	⅞″ Rivet
1	1⅝	1¾	5	$3\frac{1}{16}$	$3\frac{5}{16}$
1½	1⅞	2	5½	3¼	3½
2	$2\frac{1}{16}$	2¼	6	3⅜	3⅝
2½	2¼	$2\frac{7}{16}$	6½	3½	3¾
3	$2\frac{7}{16}$	2⅝	7	3⅝	3⅞
3½	$2\frac{9}{16}$	$2\frac{13}{16}$	7½	3¾	4
4	$2\frac{13}{16}$	3	8	3⅞	4⅛
4½	$2\frac{15}{16}$	$3\frac{3}{16}$	8½	4	4¼

y = diameter of rivet + ⅛″

$a - y = \sqrt{a^2 + b^2} - 2y$ $\qquad$ $a^1 - 2y = \sqrt{a^2 + b^2} - 3y$

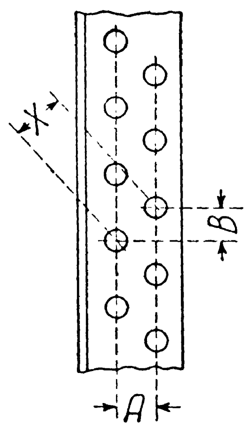
$b = \sqrt{2ay + y^2}$ $\qquad$ $b = \sqrt{2ay + y^2}$

a = sum of gages minus thickness of angle.
⅝″ rivets, can be taken at ⅛″ less than for ¾″ rivets.
1″ rivets, can be taken at ⅛″ more than for ⅞″ rivets.

TABLE 110.

Standards for Riveting.

Distance ¢ to ¢ of Staggered Rivets.



Values of X for varying values of A and B.

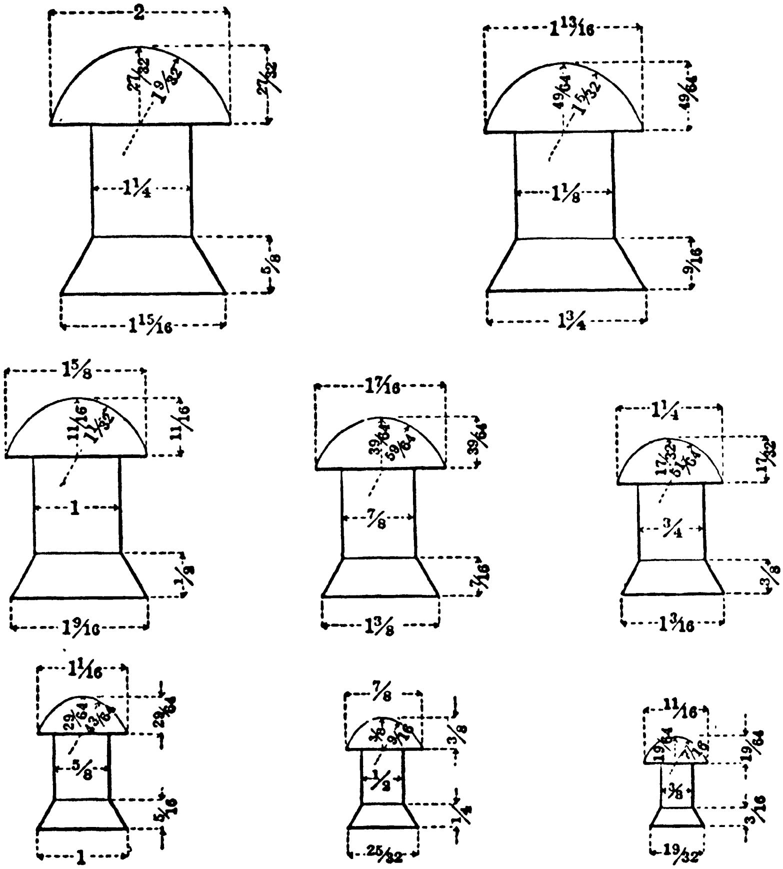
Values of B	Values of A: 7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8	1 3/4	1 7/8	2	2 1/8	2 1/4	2 3/8	2 1/2
1 1/8	1 7/16	1 1/2	1 9/16	1 11/16	1 3/4	1 7/8	2	2 1/16	2 3/16	2 5/16	2 3/8	2 1/2	2 5/8	2 3/4
1 1/4	1 9/16	1 5/8	1 11/16	1 3/4	1 7/8	1 15/16	2 1/16	2 1/8	2 1/4	2 3/8	2 7/16	2 9/16	2 11/16	2 13/16
1 3/8	1 5/8	1 11/16	1 3/4	1 7/8	1 15/16	2	2 1/8	2 3/16	2 5/16	2 7/16	2 1/2	2 5/8	2 3/4	2 7/8
1 1/2	1 3/4	1 13/16	1 7/8	1 15/16	2	2 1/8	2 3/16	2 5/16	2 3/8	2 1/2	2 5/8	2 11/16	2 13/16	2 15/16
1 5/8	1 7/8	1 7/8	2	2 1/16	2 1/8	2 3/16	2 5/16	2 3/8	2 1/2	2 9/16	2 11/16	2 3/4	2 7/8	3
1 3/4	1 15/16	2	2 1/16	2 1/8	2 3/16	2 5/16	2 3/8	2 7/16	2 9/16	2 5/8	2 3/4	2 7/8	2 15/16	3 1/16
1 7/8	2 1/16	2 1/8	2 3/16	2 1/4	2 5/16	2 3/8	2 1/2	2 9/16	2 5/8	2 3/4	2 13/16	2 15/16	3	3 1/8
2	2 3/16	2 1/4	2 5/16	2 3/8	2 7/16	2 1/2	2 9/16	2 5/8	2 3/4	2 13/16	2 15/16	3	3 1/8	3 3/16
2 1/8	2 5/16	2 5/16	2 3/8	2 7/16	2 1/2	2 5/8	2 11/16	2 3/4	2 13/16	2 15/16	3	3 1/16	3 3/16	3 1/4
2 1/4	2 7/16	2 7/16	2 1/2	2 9/16	2 5/8	2 11/16	2 3/4	2 7/8	2 15/16	3	3 1/16	3 3/16	3 1/4	3 3/8
2 3/8	2 1/2	2 9/16	2 5/8	2 11/16	2 3/4	2 13/16	2 7/8	2 15/16	3	3 1/8	3 3/16	3 1/4	3 3/8	3 7/16
2 1/2	2 5/8	2 11/16	2 3/4	2 13/16	2 7/8	2 15/16	3	3 1/16	3 1/8	3 3/16	3 1/4	3 3/8	3 7/16	3 9/16

NOTE:—Values below or to the right of upper zigzag line are large enough for 5/8" Riv.
" " " " " " " second " " " " " " 3/4" "
" " " " " " " lower " " " " " " 7/8" "

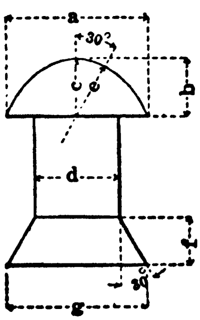
TABLE 111.

STANDARDS FOR RIVETING.

Dimensions in Inches



GENERAL FORMULAS FOR PROPORTIONS OF RIVETS, IN INCHES



Full driven head, diameter, $a = 1.5\,d + \frac{1}{8}''$

" " " depth, $b = 0.425\,a$

" " " radius, $c = b$

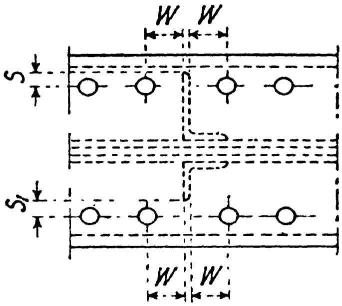
" " " radius, $e = 1.5\,b$

Countersunk head, depth, $f = 0.5\,d$

" " " diameter, $g = 1.577\,d$

TABLE 112.

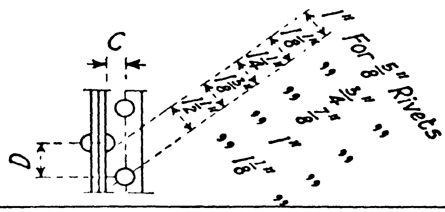
Standards for Riveting.



Clearance for Cover Plate Riveting

S	1/2"	1"	1 1/2"	2"	2 1/2"	3"	3 1/2"	4"	4 1/2"	5"	5 1/2"	6"
W	2 1/2	2 5/8	2 3/4	2 3/4	2 7/8	2 7/8	3	3 1/8	3 1/8	3 1/4	3 1/4	3 3/8
S_1	0	1/2	1	1 1/2	2	2 1/2						
W	2 1/2	2 1/4	2 1/8	2	1 1/2	0						

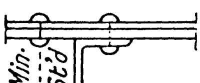
Minimum Stagger for Rivets



Value of D in Inches.

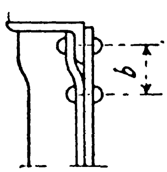
Riv. \ C	1 1/8"	1 3/16"	1 1/4"	1 5/16"	1 3/8"	1 7/16"	1 1/2"	1 9/16"	1 5/8"	1 11/16"	1 3/4"	1 13/16"	1 7/8"	1 15/16"	2 1/16"	2 3/16"	2 5/16"
5/8"	15/16	7/8	13/16	11/16	1/2	5/16	0										
3/4	1 1/4	1 3/16	1 1/8	1 1/16	15/16	7/8	3/4	9/16	3/8	0							
7/8	1 1/2	1 7/16	1 3/8	1 5/16	1 1/4	1 3/16	1 1/8	1	15/16	13/16	5/8	7/16	0				
1	1 13/16	1 3/4	1 11/16	1 5/8	1 9/16	1 1/2	1 7/16	1 3/8	1 5/16	1 3/16	1 1/8	1	7/8	3/4	0		
1 1/8	2 1/16	2	1 15/16	1 15/16	1 7/8	1 13/16	1 3/4	1 11/16	1 5/8	1 9/16	1 1/2	1 3/8	1 5/16	1 1/4	1	11/16	0

Clearance for Web Riveting



7/8"	1 1/8"	For 5/8" Rivets
1"	1 1/4"	" 3/4" "
1 1/8"	1 3/8"	" 7/8" "
1 1/4"	1 1/2"	" 1" "
1 3/8"	1 5/8"	" 1 1/8" "

Rivets in Crimped Angles



Distance "b" should be 1 1/2" plus thickness of chord angles, but never less than 2".

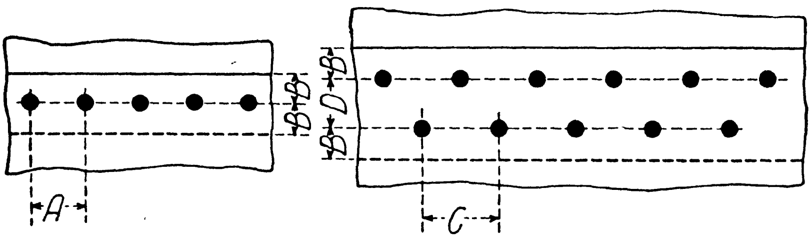
Standard Rivet Dies

2"	For 5/8" Rivets
2 1/4"	" 3/4" "
2 1/2"	" 7/8" "
2 3/4"	" 1" "
3"	" 1 1/8" "

TABLE 113.

STANDARDS FOR RIVETING.

STANDARD RIVET SPACING FOR CAULKING



THICKNESS OF PLATE	$\frac{3}{8}''$ RIVETS				$\frac{1}{2}''$ RIVETS				$\frac{5}{8}''$ RIVETS				$\frac{3}{4}''$ RIVETS				$\frac{7}{8}''$ RIVETS			
	A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D
$\frac{1}{8}''$	$1\frac{1}{4}$	$\frac{5}{8}$	2	1																
$\frac{3}{16}''$	$1\frac{1}{2}$	$\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{1}{4}$	$1\frac{5}{8}$	$\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{3}{8}$												
$\frac{1}{4}''$	$1\frac{1}{2}$	$\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{1}{4}$	$1\frac{3}{4}$	$\frac{7}{8}$	$2\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{7}{8}$	1	$2\frac{5}{8}$	$1\frac{5}{8}$	2	$1\frac{1}{8}$	$2\frac{3}{4}$	$1\frac{3}{4}$				
$\frac{5}{16}''$					$1\frac{3}{4}$	$\frac{7}{8}$	$2\frac{1}{2}$	$1\frac{1}{2}$	2	1	$2\frac{3}{4}$	$1\frac{3}{4}$	$2\frac{1}{8}$	$1\frac{1}{8}$	$2\frac{7}{8}$	$1\frac{7}{8}$				
$\frac{3}{8}''$					$1\frac{7}{8}$	1	$2\frac{5}{8}$	$1\frac{1}{2}$	2	1	$2\frac{3}{4}$	$1\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{1}{8}$	3	2	$2\frac{3}{8}$	$1\frac{1}{4}$	$3\frac{1}{8}$	$2\frac{1}{8}$
$\frac{7}{16}''$									$2\frac{1}{8}$	1	$2\frac{7}{8}$	$1\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{1}{8}$	3	2	$2\frac{3}{8}$	$1\frac{3}{8}$	$3\frac{1}{4}$	$2\frac{1}{4}$
$\frac{1}{2}''$									$2\frac{1}{4}$	$1\frac{1}{8}$	3	$1\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{1}{4}$	$3\frac{1}{8}$	$2\frac{1}{8}$	$2\frac{1}{2}$	$1\frac{1}{2}$	$3\frac{1}{4}$	$2\frac{1}{4}$
$\frac{5}{8}''$													$2\frac{1}{2}$	$1\frac{1}{4}$	$3\frac{1}{4}$	$2\frac{1}{8}$	$2\frac{5}{8}$	$1\frac{1}{2}$	$3\frac{3}{8}$	$2\frac{1}{4}$
$\frac{3}{4}''$																				

TABLE 114

Shearing and Bearing Value of Rivets

Values above or to right of upper zigzag lines are greater than double shear.
Values below or to left of lower zigzag lines are less than single shear.

Rivet Diam., In.	Rivet Area, Sq. In.	Single Shear at 6 000 Pounds	Bearing Value for Different Thicknesses of Plate at 12 000 Lbs. Per Square Inch.												
			1/4″	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
1/2	.196	1 180	1 500	1 880	2 250	2 630	3 000	.....	.....	.....	.....	.....	.....	.....	.....
5/8	.307	1 840	1 880	2 340	2 810	3 280	3 750	4 220	4 690	.....	.....	.....	.....	.....	.....
3/4	.442	2 650	2 250	2 810	3 380	3 940	4 500	5 060	5 630	6 190	6 750	.....	.....	.....	.....
7/8	.601	3 610	2 630	3 280	3 910	4 590	5 250	5 910	6 560	7 220	7 880	8 530	9 190	9 840	.....
1	.785	4 710	3 000	3 750	4 500	5 260	6 000	6 750	7 500	8 250	9 000	9 750	10 500	11 250	12 000

Rivet Diam., In.	Rivet Area, Sq. In.	Single Shear at 7 500 Pounds	Bearing Value for Different Thicknesses of Plate at 15 000 Lbs. Per Square Inch												
			1/4″	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
1/2	.196	1 470	1 880	2 340	2 810	3 280	3 750	.....	.....	.....	.....	.....	.....	.....	.....
5/8	.307	2 300	2 340	2 930	3 520	4 100	4 690	5 270	5 860	.....	.....	.....	.....	.....	.....
3/4	.442	3 310	2 810	3 520	4 220	4 920	5 630	6 330	7 030	7 730	8 440	.....	.....	.....	.....
7/8	.601	4 510	3 280	4 100	4 920	5 740	6 560	7 380	8 200	9 020	9 840	10 660	11 480	12 300	.....
1	.785	5 890	3 750	4 690	5 630	6 560	7 500	8 440	9 380	10 310	11 250	12 190	13 130	14 060	15 000

Rivet Diam., In.	Rivet Area, Sq. In.	Single Shear at 10 000 Pounds	Bearing Value for Different Thicknesses of Plate at 20 000 Lbs. Per Square Inch												
			1/4″	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
1/2	.196	1 960	2 500	3 130	3 750	4 380	5 000	.....	.....	.....	.....	.....	.....	.....	.....
5/8	.307	3 070	3 130	3 910	4 690	5 470	6 250	7 030	7 810	.....	.....	.....	.....	.....	.....
3/4	.442	4 420	3 750	4 690	5 630	6 560	7 500	8 440	9 380	10 310	11 250	.....	.....	.....	.....
7/8	.601	6 010	4 380	5 470	6 560	7 660	8 750	9 840	10 940	12 030	13 130	14 220	15 310	16 410	.....
1	.785	7 850	5 000	6 250	7 500	8 750	10 000	11 250	12 500	13 750	15 000	16 250	17 500	18 750	20 000

Rivet Diam., In.	Rivet Area, Sq. In.	Single Shear at 11 000 Pounds	Bearing Value for Different Thicknesses of Plate at 22 000 Lbs. Per Square Inch												
			1/4″	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
1/2	.196	2 160	2 750	3 440	4 130	4 810	5 500	.....	.....	.....	.....	.....	.....	.....	.....
5/8	.307	3 370	3 440	4 300	5 160	6 020	6 880	7 730	8 590	.....	.....	.....	.....	.....	.....
3/4	.442	4 860	4 130	5 160	6 190	7 220	8 250	9 280	10 310	11 340	12 380	.....	.....	.....	.....
7/8	.601	6 610	4 810	6 020	7 220	8 420	9 630	10 830	12 030	13 230	14 440	15 640	16 840	18 050	.....
1	.785	8 640	5 500	6 880	8 250	9 630	11 000	12 380	13 750	15 130	16 500	17 880	19 250	20 630	22 000

Rivet Diam., In.	Rivet Area, Sq. In.	Single Shear at 12 000 Pounds	Bearing Value for Different Thicknesses of Plate at 24 000 Lbs. Per Square Inch												
			1/4″	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″	15/16″	1″
1/2	.196	2 360	3 000	3 750	4 500	5 250	6 000	.....	.....	.....	.....	.....	.....	.....	.....
5/8	.307	3 680	3 750	4 690	5 630	6 560	7 500	8 440	9 380	.....	.....	.....	.....	.....	.....
3/4	.442	5 300	4 500	5 630	6 750	7 880	9 000	10 130	11 250	12 380	13 500	.....	.....	.....	.....
7/8	.601	7 220	5 250	6 560	7 880	9 190	10 500	11 810	13 130	14 440	15 750	17 060	18 380	.....	..
1	.785	9 420	6 000	7 500	9 000	10 500	12 000	13 500	15 000	16 500	18 000	19 500	21 000	22 500	24 000

TABLE 115

Multiplication Table for Rivet Spacing

Spaces	Pitch of Rivets in Inches															Spaces
	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8	1 3/4	1 7/8	2	2 1/8	2 1/4	2 3/8	2 1/2	2 5/8	2 3/4	2 7/8	
1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	....	.....	1
2	– 2 1/4	– 2 1/2	– 2 3/4	– 3	– 3 1/4	– 3 1/2	– 3 3/4	– 4	– 4 1/4	– 4 1/2	– 4 3/4	– 5	– 5 1/4	– 5 1/2	– 5 3/4	2
3	– 3 3/8	– 3 3/4	– 4 1/8	– 4 1/2	– 4 7/8	– 5 1/4	– 5 5/8	– 6	– 6 3/8	– 6 3/4	– 7 1/8	– 7 1/2	– 7 7/8	– 8 1/4	– 8 5/8	3
4	– 4 1/2	– 5	– 5 1/2	– 6	– 6 1/2	– 7	– 7 1/2	– 8	– 8 1/2	– 9	– 9 1/2	–10	–10 1/2	–11	–11 1/2	4
5	– 5 5/8	– 6 1/4	– 6 7/8	– 7 1/2	– 8 1/8	– 8 3/4	– 9 3/8	–10	–10 5/8	–11 1/4	–11 7/8	1– 0 1/2	1– 1 1/8	1– 1 3/4	1– 2 3/8	5
6	– 6 3/4	– 7 1/2	– 8 1/4	– 9	– 9 3/4	–10 1/2	–11 1/4	1– 0	1– 0 3/4	1– 1 1/2	1– 2 1/4	1– 3	1– 3 3/4	1– 4 1/2	1– 5 1/4	6
7	– 7 7/8	– 8 3/4	– 9 5/8	–10 1/2	–11 3/8	1– 0 1/4	1– 1 1/8	1– 2	1– 2 7/8	1– 3 3/4	1– 4 5/8	1– 5 1/2	1– 6 3/8	1– 7 1/4	1– 8 1/8	7
8	– 9	–10	–11	1– 0	1– 1	1– 2	1– 3	1– 4	1– 5	1– 6	1– 7	1– 8	1– 9	1–10	1–11	8
9	–10 1/8	–11 1/4	1– 0 3/8	1– 1 1/2	1– 2 5/8	1– 3 3/4	1– 4 7/8	1– 6	1– 7 1/8	1– 8 1/4	1– 9 3/8	1–10 1/2	1–11 5/8	2– 0 3/4	2– 1 7/8	9
10	–11 1/4	1– 0 1/2	1– 1 3/4	1– 3	1– 4 1/4	1– 5 1/2	1– 6 3/4	1– 8	1– 9 1/4	1–10 1/2	1–11 3/4	2– 1	2– 2 1/4	2– 3 1/2	2– 4 3/4	10
11	1– 0 3/8	1– 1 3/4	1– 3 1/8	1– 4 1/2	1– 5 7/8	1– 7 1/4	1– 8 5/8	1–10	1–11 3/8	2– 0 3/4	2– 2 1/8	2– 3 1/2	2– 4 7/8	2– 6 1/4	2– 7 5/8	11
12	1– 1 1/2	1– 3	1– 4 1/2	1– 6	1– 7 1/2	1– 9	1–10 1/2	2– 0	2– 1 1/2	2– 3	2– 4 1/2	2– 6	2– 7 1/2	2– 9	2–10 1/2	12
13	1– 2 5/8	1– 4 1/4	1– 5 7/8	1– 7 1/2	1– 9 1/8	1–10 3/4	2– 0 3/8	2– 2	2– 3 5/8	2– 5 1/4	2– 6 7/8	2– 8 1/2	2–10 1/8	2–11 3/4	3– 1 3/8	13
14	1– 3 3/4	1– 5 1/2	1– 7 1/4	1– 9	1–10 3/4	2– 0 1/2	2– 2 1/4	2– 4	2– 5 3/4	2– 7 1/2	2– 9 1/4	2–11	3– 0 3/4	3– 2 1/2	3– 4 1/4	14
15	1– 4 7/8	1– 6 3/4	1– 8 5/8	1–10 1/2	2– 0 3/8	2– 2 1/4	2– 4 1/8	2– 6	2– 7 7/8	2– 9 3/4	2–11 5/8	3– 1 1/2	3– 3 3/8	3– 5 1/4	3– 7 1/8	15
16	1– 6	1– 8	1–10	2– 0	2– 2	2– 4	2– 6	2– 8	2–10	3– 0	3– 2	3– 4	3– 6	3– 8	3–10	16
17	1– 7 1/8	1– 9 1/4	1–11 3/8	2– 1 1/2	2– 3 5/8	2– 5 3/4	2– 7 7/8	2–10	3– 0 1/8	3– 2 1/4	3– 4 3/8	3– 6 1/2	3– 8 5/8	3–10 3/4	4– 0 7/8	17
18	1– 8 1/4	1–10 1/2	2– 0 3/4	2– 3	2– 5 1/4	2– 7 1/2	2– 9 3/4	3– 0	3– 2 1/4	3– 4 1/2	3– 6 3/4	3– 9	3–11 1/4	4– 1 1/2	4– 3 3/4	18
19	1– 9 3/8	1–11 3/4	2– 2 1/8	2– 4 1/2	2– 6 7/8	2– 9 1/4	2–11 5/8	3– 2	3– 4 3/8	3– 6 3/4	3– 9 1/8	3–11 1/2	4– 1 7/8	4– 4 1/4	4– 6 5/8	19
20	1–10 1/2	2– 1	2– 3 1/2	2– 6	2– 8 1/2	2–11	3– 1 1/2	3– 4	3– 6 1/2	3– 9	3–11 1/2	4– 2	4– 4 1/2	4– 7	4– 9 1/2	20
21	1–11 5/8	2– 2 1/4	2– 4 7/8	2– 7 1/2	2–10 1/8	3– 0 3/4	3– 3 3/8	3– 6	3– 8 5/8	3–11 1/4	4– 1 7/8	4– 4 1/2	4– 7 1/8	4– 9 3/4	5– 0 3/8	21
22	2– 0 3/4	2– 3 1/2	2– 6 1/4	2– 9	2–11 3/4	3– 2 1/2	3– 5 1/4	3– 8	3–10 3/4	4– 1 1/2	4– 4 1/4	4– 7	4– 9 3/4	5– 0 1/2	5– 3 1/4	22
23	2– 1 7/8	2– 4 3/4	2– 7 5/8	2–10 1/2	3– 1 3/8	3– 4 1/4	3– 7 1/8	3–10	4– 0 7/8	4– 3 3/4	4– 6 5/8	4– 9 1/2	5– 0 3/8	5– 3 1/4	5– 6 1/8	23
24	2– 3	2– 6	2– 9	3– 0	3– 3	3– 6	3– 9	4– 0	4– 3	4– 6	4– 9	5– 0	5– 3	5– 6	5– 9	24
25	2– 4 1/8	2– 7 1/4	2–10 3/8	3– 1 1/2	3– 4 5/8	3– 7 3/4	3–10 7/8	4– 2	4– 5 1/8	4– 8 1/4	4–11 3/8	5– 2 1/2	5– 5 5/8	5– 8 3/4	5–11 7/8	25
26	2– 5 1/4	2– 8 1/2	2–11 3/4	3– 3	3– 6 1/4	3– 9 1/2	4– 0 3/4	4– 4	4– 7 1/4	4–10 1/2	5– 1 3/4	5– 5	5– 8 1/4	5–11 1/2	6– 2 3/4	26
27	2– 6 3/8	2– 9 3/4	3– 1 1/8	3– 4 1/2	3– 7 7/8	3–11 1/4	4– 2 5/8	4– 6	4– 9 3/8	5– 0 3/4	5– 4 1/8	5– 7 1/2	5–10 7/8	6– 2 1/4	6– 5 5/8	27
28	2– 7 1/2	2–11	3– 2 1/2	3– 6	3– 9 1/2	4– 1	4– 4 1/2	4– 8	4–11 1/2	5– 3	5– 6 1/2	5–10	6– 1 1/2	6– 5	6– 8 1/2	28
29	2– 8 5/8	3– 0 1/4	3– 3 7/8	3– 7 1/2	3–11 1/8	4– 2 3/4	4– 6 3/8	4–10	5– 1 5/8	5– 5 1/4	5– 8 7/8	6– 0 1/2	6– 4 1/8	6– 7 3/4	6–11 3/8	29
30	2– 9 3/4	3– 1 1/2	3– 5 1/4	3– 9	4– 0 3/4	4– 4 1/2	4– 8 1/4	5– 0	5– 3 3/4	5– 7 1/2	5–11 1/4	6– 3	6– 6 3/4	6–10 1/2	7– 2 1/4	30
Spaces	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8	1 3/4	1 7/8	2	2 1/8	2 1/4	2 3/8	2 1/2	2 5/8	2 3/4	2 7/8	Spaces
	Pitch of Rivets in Inches															

TABLE 115.—*Continued*

Multiplication Table for Rivet Spacing

Spaces	Pitch of Rivets in Inches															Spaces
	3	3⅛	3¼	3⅜	3½	3¾	4	4¼	4½	4¾	5	5¼	5½	5¾	6	
1	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	1
2	-6	- 6¼	- 6½	- 6¾	- 7	- 7½	-8	- 8½	- 9	- 9½	-10	-10½	-11	-11½	1-0	2
3	-9	- 9⅜	- 9¾	-10⅛	-10½	-11¼	1-0	1- 0¾	1- 1½	1- 2¼	1- 3	1- 3¾	1- 4½	1- 5¼	1-6	3
4	1-0	1- 0½	1- 1	1- 1½	1- 2	1- 3	1-4	1- 5	1- 6	1- 7	1- 8	1- 9	1-10	1-11	2-0	4
5	1-3	1- 3⅝	1- 4¼	1- 4⅞	1- 5½	1- 6¾	1-8	1- 9¼	1-10½	1-11¾	2- 1	2- 2¼	2- 3½	2- 4¾	2-6	5
6	1-6	1- 6¾	1- 7½	1- 8¼	1- 9	1-10½	2-0	2- 1½	2- 3	2- 4½	2- 6	2- 7½	2- 9	2-10½	3-0	6
7	1-9	1- 9⅞	1-10¾	1-11⅝	2- 0½	2- 2¼	2-4	2- 5¾	2- 7½	2- 9¼	2-11	3- 0¾	3- 2½	3- 4¼	3-6	7
8	2-0	2- 1	2- 2	2- 3	2- 4	2- 6	2-8	2-10	3- 0	3- 2	3- 4	3- 6	3- 8	3-10	4-0	8
9	2-3	2- 4⅛	2- 5¼	2- 6⅜	2- 7½	2- 9¾	3-0	3- 2¼	3- 4½	3- 6¾	3- 9	3-11¼	4- 1½	4- 3¾	4-6	9
10	2-6	2- 7¼	2- 8½	2- 9¾	2-11	3- 1½	3-4	3- 6½	3- 9	3-11½	4- 2	4- 4½	4- 7	4- 9½	5-0	10
11	2-9	2-10⅜	2-11¾	3- 1⅛	3- 2½	3- 5¼	3-8	3-10¾	4- 1½	4- 4¼	4- 7	4- 9¾	5- 0½	5- 3¼	5-6	11
12	3-0	3- 1½	3- 3	3- 4½	3- 6	3- 9	4-0	4- 3	4- 6	4- 9	5- 0	5- 3	5- 6	5- 9	6-0	12
13	3-3	3- 4⅝	3- 6¼	3- 7⅞	3- 9½	4- 0¾	4-4	4- 7¼	4-10½	5- 1¾	5- 5	5- 8¼	5-11½	6- 2¾	6-6	13
14	3-6	3- 7¾	3- 9½	3-11¼	4- 1	4- 4½	4-8	4-11½	5- 3	5- 6½	5-10	6- 1½	6- 5	6- 8½	7-0	14
15	3-9	3-10⅞	4- 0¾	4- 2⅝	4- 4½	4- 8¼	5-0	5- 3¾	5- 7½	5-11¼	6- 3	6- 6¾	6-10½	7- 2¼	7-6	15
16	4-0	4- 2	4- 4	4- 6	4- 8	5- 0	5-4	5- 8	6- 0	6- 4	6- 8	7- 0	7- 4	7- 8	8-0	16
17	4-3	4- 5⅛	4- 7¼	4- 9⅜	4-11½	5- 3¾	5-8	6- 0¼	6- 4½	6- 8¾	7- 1	7- 5¼	7- 9½	8- 1¾	8-6	17
18	4-6	4- 8¼	4-10½	5- 0¾	5- 3	5- 7½	6-0	6- 4½	6- 9	7- 1½	7- 6	7-10½	8- 3	8- 7½	9-0	18
19	4-9	4-11⅜	5- 1¾	5- 4⅛	5- 6½	5-11¼	6-4	6- 8¾	7- 1½	7- 6¼	7-11	8- 3¾	8- 8½	9- 1¼	9-6	19
20	5-0	5- 2½	5- 5	5- 7½	5-10	6- 3	6-8	7- 1	7- 6	7-11	8- 4	8- 9	9- 2	9- 7	10-0	20
21	5-3	5- 5⅝	5- 8¼	5-10⅞	6- 1½	6- 6¾	7-0	7- 5¼	7-10½	8- 3¾	8- 9	9- 2¼	9- 7½	10- 0¾	10-6	21
22	5-6	5- 8¾	5-11½	6- 2¼	6- 5	6-10½	7-4	7- 9½	8- 3	8- 8½	9- 2	9- 7½	10- 1	10- 6½	11-0	22
23	5-9	5-11⅞	6- 2¾	6- 5⅝	6- 8½	7- 2¼	7-8	8- 1¾	8- 7½	9- 1¼	9- 7	10- 0¾	10- 6½	11- 0¼	11-6	23
24	6-0	6- 3	6- 6	6- 9	7- 0	7- 6	8-0	8- 6	9- 0	9- 6	10- 0	10- 6	11- 0	11- 6	12-0	24
25	6-3	6- 6⅛	6- 9¼	7- 0⅜	7- 3½	7- 9¾	8-4	8-10¼	9- 4½	9-10¾	10- 5	10-11¼	11- 5½	11-11¾	12-6	25
26	6-6	6- 9¼	7- 0½	7- 3¾	7- 7	8- 1½	8-8	9- 2½	9- 9	10- 3½	10-10	11- 4½	11-11	12- 5½	13-0	26
27	6-9	7- 0⅜	7- 3¾	7- 7⅛	7-10½	8- 5¼	9-0	9- 6¾	10- 1½	10- 8¼	11- 3	11- 9¾	12- 4½	12-11¼	13-6	27
28	7-0	7- 3½	7- 7	7-10½	8- 2	8- 9	9-4	9-11	10- 6	11- 1	11- 8	12- 3	12-10	13- 5	14-0	28
29	7-3	7- 6⅝	7-10¼	8- 1⅞	8- 5½	9- 0¾	9-8	10- 3¼	10-10½	11- 5¾	12- 1	12- 8¼	13- 3½	13-10¾	14-6	29
30	7-6	7- 9¾	8- 1½	8- 5¼	8- 9	9- 4½	10-0	10- 7½	11- 3	11-10½	12- 6	13- 1½	13- 9	14- 4½	15-0	30
Spaces	3	3⅛	3¼	3⅜	3½	3¾	4	4¼	4½	4¾	5	5¼	5½	5¾	6	Spaces
	Pitch of Rivets in Inches															

TABLE 116.

Areas to be Deducted for Rivet Holes, Maximum Rivets, and Rivet Spacing.

Areas in Square Inches, to be Deducted from Riveted Plates or Shapes to Obtain Net Areas.

Thickness of Plates.	Diameter of Hole in Inches (Diam. of Rivet + 1/8").																
Inches.	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	1 3/16	1 1/4
1/4	.06	.08	.09	.11	.13	.14	.16	.17	.19	.20	.22	.23	.25	.27	.28	.30	.31
5/16	.08	.10	.12	.14	.16	.18	.20	.21	.23	.25	.27	.29	.31	.33	.35	.37	.39
3/8	.09	.12	.14	.16	.19	.21	.23	.26	.28	.30	.33	.35	.38	.40	.42	.45	.47
7/16	.11	.14	.16	.19	.22	.25	.27	.30	.33	.36	.38	.41	.44	.46	.49	.52	.55
1/2	.13	.16	.19	.22	.25	.28	.31	.34	.38	.41	.44	.47	.50	.53	.56	.59	.63
9/16	.14	.18	.21	.25	.28	.32	.35	.39	.42	.46	.49	.53	.56	.60	.63	.67	.70
5/8	.16	.20	.23	.27	.31	.35	.39	.43	.47	.51	.55	.59	.63	.66	.70	.74	.78
11/16	.17	.21	.26	.30	.34	.39	.43	.47	.52	.56	.60	.64	.69	.73	.77	.82	.86
3/4	.19	.23	.28	.33	.38	.42	.47	.52	.56	.61	.66	.70	.75	.80	.84	.89	.94
13/16	.20	.25	.30	.36	.41	.46	.51	.56	.61	.66	.71	.76	.81	.86	.91	.96	1.02
7/8	.22	.27	.33	.38	.44	.49	.55	.60	.66	.71	.77	.82	.88	.93	.98	1.04	1.09
15/16	.23	.29	.35	.41	.47	.53	.59	.64	.70	.76	.82	.88	.94	1.00	1.05	1.11	1.17
1	.25	.31	.38	.44	.50	.56	.63	.69	.75	.81	.88	.94	1.00	1.06	1.13	1.19	1.25
1 1/16	.27	.33	.40	.46	.53	.60	.66	.73	.80	.86	.93	1.00	1.06	1.13	1.20	1.26	1.33
1 1/8	.28	.35	.42	.49	.56	.63	.70	.77	.84	.91	.98	1.05	1.13	1.20	1.27	1.34	1.41
1 3/16	.30	.37	.45	.52	.59	.67	.74	.82	.89	.96	1.04	1.11	1.19	1.26	1.34	1.41	1.48
1 1/4	.31	.39	.47	.55	.63	.70	.78	.86	.94	1.02	1.09	1.17	1.25	1.33	1.41	1.48	1.56
1 5/16	.33	.41	.49	.57	.66	.74	.82	.90	.98	1.07	1.15	1.23	1.31	1.39	1.48	1.56	1.64
1 3/8	.34	.43	.52	.60	.69	.77	.86	.95	1.03	1.12	1.20	1.29	1.38	1.46	1.55	1.63	1.72
1 7/16	.36	.45	.54	.63	.72	.81	.90	.99	1.08	1.17	1.26	1.35	1.44	1.53	1.62	1.71	1.80
1 1/2	.38	.47	.56	.66	.75	.84	.94	1.03	1.13	1.22	1.31	1.41	1.50	1.59	1.69	1.78	1.88
1 9/16	.39	.49	.59	.68	.78	.88	.98	1.07	1.17	1.27	1.37	1.46	1.56	1.66	1.76	1.86	1.95
1 5/8	.41	.51	.61	.71	.81	.91	1.02	1.12	1.22	1.32	1.42	1.52	1.63	1.73	1.83	1.93	2.03
1 11/16	.42	.53	.63	.74	.84	.95	1.05	1.16	1.27	1.37	1.47	1.58	1.69	1.79	1.90	2.00	2.11
1 3/4	.44	.55	.66	.77	.88	.98	1.09	1.20	1.31	1.42	1.53	1.64	1.75	1.86	1.97	2.08	2.19
1 13/16	.45	.57	.68	.79	.91	1.02	1.13	1.25	1.36	1.47	1.59	1.70	1.81	1.93	2.04	2.15	2.27
1 7/8	.47	.59	.70	.82	.94	1.05	1.17	1.29	1.41	1.52	1.64	1.76	1.88	1.99	2.11	2.23	2.34
1 15/16	.48	.61	.73	.85	.97	1.09	1.21	1.33	1.45	1.57	1.70	1.82	1.94	2.06	2.18	2.30	2.42
2	.50	.63	.75	.88	1.00	1.13	1.25	1.38	1.50	1.63	1.75	1.88	2.00	2.13	2.25	2.38	2.50

Maximum Rivet in Leg of Angles or Flange of Beams and Channels.

Leg of Angle	3/4	1	1 1/4	1 3/8	1 1/2	1 3/4	2	2 1/2	3	3 1/2	4	5	6	7	8
Max. Rivet	1/4	1/4	3/8	3/8	3/8	1/2	5/8	3/4	7/8	7/8	7/8	7/8	7/8	1	1 1/8
Depth of Beam	3	4	5	6	7	8	9	10	12	15	18	20	24		
Max. Rivet	3/8	1/2	1/2	5/8	5/8	3/4	3/4	3/4	3/4	3/4	7/8	7/8	7/8		
Depth of Channel	3	4	5	6	7	8	9	10	12	15					
Max. Rivet	1/2	1/2	1/2	5/8	5/8	3/4	3/4	3/4	7/8	7/8					

Rivet Spacing in Inches.

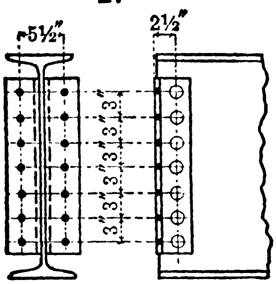
Size of Rivet.	Minimum Pitch.		Max. Pitch in Line of Stress.				Min. Edge Dist		Max. Edge Dist.
	Allowed.	Preferred.	At Ends of Comp. Mem.	Bridges.		Bld'gs.	Sheared.	Rolled.	
1/2"	1 1/2	1 3/4	2	4	16 x thickness of thinnest outside plate.	6	1	7/8	8 x thickness of plate.
5/8"	1 7/8	2	2 1/2	4 1/2		"	1 1/8	1	
3/4"	2 1/4	2 1/2	3	5		"	1 1/4	1 1/8	
7/8"	2 5/8	3	3 1/2	6		"	1 1/2	1 1/4	

TABLE 117*a*.

STANDARD CONNECTIONS FOR BEAMS AND CHANNELS.

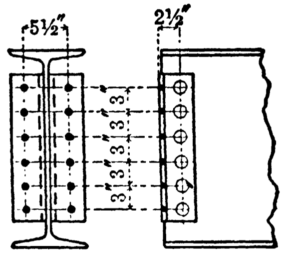
AMERICAN BRIDGE COMPANY.

27″



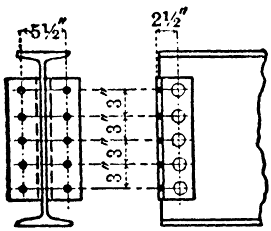
2 Ls 4″ x 4″ x ½″ x 1′-8½″
Weight 46 lbs.

24″



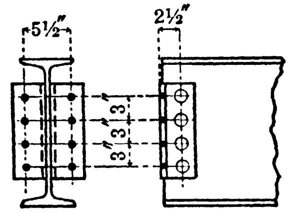
2 Ls 4″ x 4″ x ½″ x 1′-5½″
Weight 39 lbs.

21″



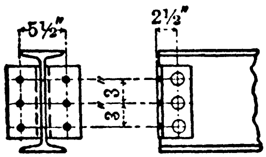
2 Ls 4″ x 4″ x ½″ x 1′-2½″
Weight 33 lbs.

20,″ 18,″ 15″



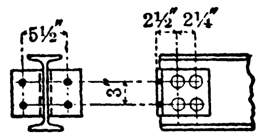
2 Ls 4″ x 4″ x 7/16″ x 0′-11½″
Weight 23 lbs.

12″



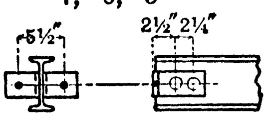
2 Ls 4″ x 4″ x 7/16″ x 0′-8½″
Weight 17 lbs.

10,″ 9,″ 8″



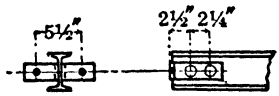
2 Ls 6″ x 4″ x ⅜″ x 0′-5½″
Weight 13 lbs.

7,″ 6,″ 5″



2 Ls 6″ x 4″ x ⅜″ x 0′-3″
Weight 7 lbs.

4,″ 3″



2 Ls 6″ x 4″ x ⅜″ x 0′-2″
Weight 5 lbs.

Rivets and bolts ¾″ diameter.

Weights given are for ¾-inch shop rivets and angle connections; about 20 per cent should be added for field rivets or bolts.

TABLE 117b.

STANDARD CONNECTIONS FOR BEAMS AND CHANNELS.

AMERICAN BRIDGE COMPANY.

I Beams		Value of Web Connection	Values of Outstanding Legs of Connection Angles					
			Field Rivets			Field Bolts		
Depth, Inches	Weight Pounds per Foot	Shop Rivets in Enclosed Bearing, Pounds	¾″ Rivets or Turned Bolts, Single Shear, Pounds	Minimum Allowable Span in Feet, Uniform Load	t, In.	¾″ Rough Bolts, Single Shear, Pounds	Minimum Allowable Span in Feet, Uniform Load	t, In.
27	90.0	82530	61900	18.9	⅝	49500	23.6	⅝
24	79.9	67500	53000	17.5	⅝	42400	21.9	⅝
	74.2	64260	53000	16.4	⅝	42400	20.4	⅝
21	60.4	48150	44200	14.2	⅝	35300	17.8	⅝
20	65.4	45000	35300	17.6	⅝	28300	22.1	⅝
18	54.7	41400	35300	13.3	⅝	28300	16.7	⅝
	48.2	34200	35300	12.8	9/16	28300	15.4	⅝
15	42.9	36900	35300	8.9	⅝	28300	11.1	⅝
	37.3	29880	35300	9.7	½	28300	10.2	9/16
12	31.8	23600	26500	8.1	9/16	21200	9.0	⅝
	27.9	19170	26500	9.2	7/16	21200	9.2	½
10	25.4	27900	17700	7.4	⅝	14100	9.2	⅝
	22.4	22680	17700	6.8	⅝	14100	8.6	⅝
9	21.8	26100	17700	5.7	⅝	14100	7.1	⅝
8	18.4	24300	17700	4.3	⅝	14100	5.4	⅝
	17.5	19800	17700	4.4	⅝	14100	5.5	⅝
7	15.3	11300	8800	6.2	⅝	7100	7.8	⅝
6	12.5	10400	8800	4.4	⅝	7100	5.5	⅝
5	10.0	9500	8800	2.9	⅝	7100	3.6	⅝
4	7.7	8600	8800	2.2	9/16	7100	2.7	⅝
3	5.7	7700	8800	1.3	½	7100	1.4	⅝

ALLOWABLE UNIT STRESS IN POUNDS PER SQUARE INCH

Single Shear	Rivets.................Shop	12000	Bearing	Rivets—enclosed.......Shop	30000
	Rivets and Turned Bolts...Field	10000		Rivets—one side.......Shop	24000
	Rough Bolts............Field	8000		Rivets and Turned Bolts, Field	20000
				Rough Bolts............Field	16000

t=Web thickness, in bearing, to develop max. allowable reactions, when beams frame opposite.

Connections are figured for bearing and shear (no moment considered).

The above values agree with tests made on beams under ordinary conditions of use.

Where web is enclosed between connection angles (enclosed bearing), values are greater because of the increased efficiency due to friction and grip.

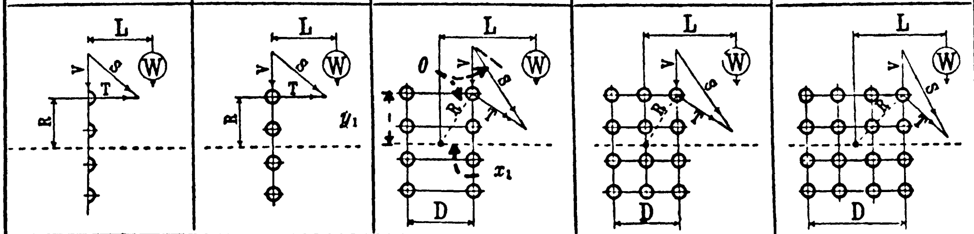
Special connections shall be used when any of the limiting conditions given above are exceeded—such as end reaction from loaded beam being greater than value of connection; shorter span with beam fully loaded; or a less thickness of web when maximum allowable reactions are used.

TABLE 118a.

Stresses in Eccentric Riveted Connections.

American Bridge Company.

VERTICAL SPACING OF RIVETS 3 INCHES.



Number of Rivets in one Vert. Row	ONE LINE	ONE LINE	TWO LINES D = 3	6	9	12	THREE LINES D = 6	8	10	12	FOUR LINES D = 9	12	15	18
		Following Table gives Values of "Q" for Various Rivet Groups.												
1	0	0	3	6	9	12	6	8	10	12	10	13	16	20
2	3	3	8	13	19	24	14	18	21	25	22	28	35	41
3	6	6	14	21	29	37	25	30	35	40	38	46	55	64
4	10	10	22	30	39	50	38	43	49	56	56	66	77	89
5	15	15	32	40	50	64	53	59	66	74	77	89	102	116
6	21	21	44	52	63	77	72	77	86	94	102	116	130	146
7	28	28	58	66	77	93	93	99	107	116	131	145	160	178
8	36	36	75	82	94	110	117	123	131	141	163	178	194	213
9	45	45	93	100	113	129	144	150	159	168	199	215	232	252
10	55	55	113	120	132	149	174	180	189	199	240	256	273	294
11	66	66	135	142	154	172	207	214	222	233	284	300	318	339
12	78	78	159	166	178	196	243	250	258	268	332	348	365	390

Stresses in Rivets in Eccentric Connections.

W = load in pounds.
L = distance from center of group to load.
R = distance from center of group to extreme rivet.
N = number of rivets in group.
$I = \Sigma d^2$ = torsional moment on rivets.
$Q = I/R$ = modulus of rivet group.
$T = M/Q$ = stress due to moment on extreme rivet.
$V = W/N$ = direct shear on extreme rivet.
S = resultant stress on extreme rivet.
$M = W.L$ = moment in in.-lb.
x_1, y_1 = coordinates of extreme rivet.
$C = W/S$ = coefficient of rivet group.

From middle figure on this page

$$S = \sqrt{V^2 + 2V \cdot T \cdot \cos\theta + T^2}$$
$$= \sqrt{V^2 + 2V \cdot T \cdot x_1/R + T^2} \qquad (1)$$

If r = allowable stress on a rivet, the safe vertical stress will be

$$f = r.V/S \qquad (2)$$

The equivalent number of rivets in direct shear will be

$$C = W/S \qquad (3)$$

Values of C for several rivet groups are given in Table 118b.

Example 1.—Stresses in standard connection for 24 in. I-beam, Table 117. Rivets in one row. N = 6. Rivet spacing 3 in. $L = 2\frac{1}{2}$ in. Now $V = W/6$. From Table 118a, $Q = 21$. $T = 2.5W/21$.
From equation (1), since $x_1 = 0$

$$S = \sqrt{(W/6)^2 + (2.5W/21)^2} = 0.2W$$
$$C = W/S = 5 \text{ rivets.}$$

Example 2.—Calculate stresses in Fig. 1, Table 118b. $L = 12$ in., $D = 7\frac{1}{2}$ in., $N = 12$. $R = \sqrt{3.75^2 + 7.5^2} = 8.4''$. $\Sigma d^2 = \Sigma x^2 + \Sigma y^2 = 12 \times 3.75^2 + 4(7.5^2 + 4.5^2 + 1.5^2) = 484$. $Q = 484/8.4 = 58$. (Interpolating in Table 118a between 6 and 9, $Q = (52 + 63)/2 = 57.5$).

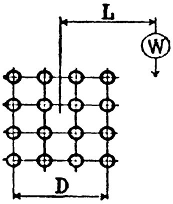
$$S = \sqrt{\left(\frac{W}{12}\right)^2 + \frac{W}{12} \times \frac{12W}{58} \times \frac{3.75}{7.50} + \left(\frac{12W}{58}\right)^2}$$
$$= .25W$$
$$C = W/S = 4 \text{ rivets.}$$

TABLE 118b.

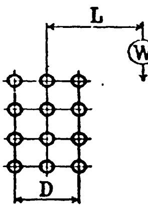
Equivalent Rivets in Eccentric Connections.

American Bridge Company.

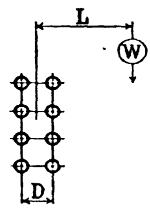
L—Dist. center of rivet group to Load W. D—Dist. between outside rivet lines. N—Total number of rivets in group.	W—Safe Load (in thousand pounds). S—Stress in one (extreme) rivet, or allowed value of one rivet. C—Coefficient given in this table.	Vertical spacing of rivets 3″. $W=SC$, $S=\frac{W}{C}$, $C=\frac{W}{S}$; $C=W$ for rivet of unit value as given below.



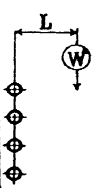
L	1½″		3″		6″		9″		12″		15″		18″		21″		24″	
N \ D	9″	18″	9″	18″	9″	18″	9″	18″	9″	18″	9″	18″	9″	18″	9″	18″	9″	18″
4	2.5	3.0	1.8	2.5	1.2	1.8	.87	1.4	.68	1.2	.57	1.0	.48	.86	.43	.77	.39	.68
8	5.2	6.1	3.9	5.0	2.5	3.6	1.9	2.7	1.5	2.3	1.3	2.0	1.1	1.8	.95	1.6	.84	1.4
12	8.4	9.8	6.4	7.7	4.1	5.7	3.2	4.5	2.5	3.6	2.1	3.2	1.8	2.7	1.6	2.3	1.4	2.2
16	12.	13.	9.3	10.	6.4	7.7	4.8	6.1	3.9	5.2	3.2	4.3	2.7	3.9	2.3	3.4	2.1	3.0
20	16.	16.	12.	14.	8.6	10.	6.6	8.0	5.2	6.6	4.3	5.7	3.6	5.0	3.2	4.3	3.0	3.9
24	20.	20.	16.	17.	11.	13.	8.6	10.	7.0	8.4	5.7	7.0	5.0	6.1	4.3	5.5	3.9	5.0
28	24.	24.	20.	20.	14.	15.	11.	12.	8.8	10.	7.3	8.9	6.4	7.7	5.5	6.8	4.8	6.1
32	28.	28.	24.	24.	17.	18.	14.	15.	11.	12.	9.1	10.	7.7	9.1	6.8	8.2	6.1	7.3
36	32.	32.	28.	28.	21.	22.	16.	17.	13.	15.	11.	12.	9.5	11.	8.4	9.8	7.5	8.6
40	36.	36.	32.	32.	25.	25.	20.	20.	16.	17.	13.	15.	12.	13.	10.	11.	9.1	10.
44	40.	40.	35.	35.	28.	28.	23.	23.	19.	20.	16.	17.	14.	15.	12.	13	11.	12.
48	44.	44.	40.	40.	32.	32.	26.	26.	21.	22.	18.	19.	16.	17.	14.	15.	12.	13.



N \ D	6″	12″	6″	12″	6″	12″	6″	12″	6″	12″	6″	12″	6″	12″	6″	12″	6″	12″
3	1.7	2.2	1.3	1.7	.75	1.2	.55	.91	.43	.75	.34	.64	.30	.55	.25	.48	.23	.43
6	3.6	4.3	2.7	3.4	1.8	2.5	1.3	1.9	1.0	1.6	.84	1.3	.73	1.1	.64	1.0	.57	.89
9	6.4	6.8	4.5	5.5	3.0	3.9	2.3	3.0	1.8	2.5	1.5	2.1	1.3	1.8	1.1	1.6	.98	1.4
12	8.9	9.3	6.8	7.5	4.5	5.5	3.4	4.3	2.7	3.4	2.3	3.0	1.9	2.5	1.6	2.3	1.5	2.0
15	12.	12.	9.3	10.	6.4	7.3	4.8	5.7	3.6	4.5	3.2	3.9	2.5	3.2	2.3	3.0	2.1	2.5
18	15.	15.	12.	12.	8.4	9.1	6.4	7.3	5.0	5.9	4.1	5.0	3.6	4.3	3.2	3.6	2.7	3.4
21	18.	18.	15.	15.	11.	11.	8.2	8.9	6.6	7.3	5.5	6.1	4.5	5.2	3.9	4.5	3.4	4.1
24	21.	21.	18.	18.	13.	14.	10.	11.	8.2	8.9	6.8	7.5	5.7	6.4	5.0	5.7	4.3	5.0
27	24.	24.	21.	21.	16.	16.	12.	13.	10.	10.	8.2	8.9	7.0	7.7	6.1	6.8	5.5	6.1
30	27.	27.	24.	24.	19.	19.	15.	15.	12.	12.	10.	11.	8.6	9.1	7.5	8.2	6.6	7.3
33	30.	30.	27.	27.	21.	22.	17.	17	14.	15.	12.	12.	10.	11.	8.9	9.3	8.0	8.4
36	33.	33.	30.	30.	24.	24.	20.	20.	16.	17.	14.	14.	12.	12.	10.	11.	9.1	9.5

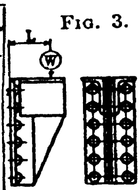


N \ D	3″	12″	3″	12″	3″	12″	3″	12″	3″	12″	3″	12″	3″	12″	3″	12″	3″	12″
2	1.0	1.6	.66	1,3	.39	1.0	.27	.80	.23	.66	.18	.57	.16	.50	.14	.43	.11	.39
4	2.5	3.2	1.8	2.5	1.1	2.1	.80	1.6	.61	1.4	.50	1.2	.43	1.0	.36	.91	.32	.82
6	4.3	4.8	3.0	4.1	2.0	3.2	1.4	2.3	1.1	2.1	.89	1.8	.75	1.6	.66	1.4	.57	1.2
8	6.1	6.6	4.5	5.5	3.0	4.3	2.2	3.4	1.7	2.7	1.4	2.3	1.2	2.1	1.0	1.9	.89	1.7
10	8.4	8.4	6.6	7.0	4.3	5.5	3.2	4.3	2.5	3.6	2.0	3.2	1.7	2.7	1.5	2.3	1.3	2.2
12	10.	10.	8.6	8.9	5.7	6.8	4.3	5.5	3.4	4.5	2.7	3.9	2.3	3.4	2.1	3.0	1.8	2.5
14	12.	12.	10.	10.	7.5	8.2	5.5	6.6	4.3	5.5	3.4	4.5	3.0	4.1	2.5	3.6	2.3	3.2
16	15.	14.	13.	12.	8.9	9.5	6.8	8.0	5.5	6.6	4.5	5.7	3.9	4.8	3.2	4.3	3.0	3.9
18	17.	16.	15.	14.	11.	11.	8.4	9.3	6.8	7.7	5.5	6.6	4.8	5.7	4.1	5.0	3.6	4.5
20	19.	18.	17.	16.	13.	13.	10.	11.	8.2	8.9	6.6	7.5	5.7	6.6	5.0	5.9	4.5	5.2
22	21.	20.	19.	18.	15.	15.	12.	12.	9.5	10.	8.0	8.9	6.8	7.7	5.9	6.8	5.2	6.1
24	23.	22.	21.	20.	17.	17.	14.	14.	11	12.	9.3	10.	8.0	8.6	7.0	7.7	6.1	6.8

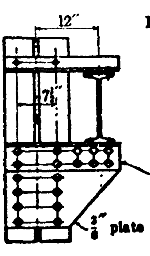


N \ L	1½″	3″	6″	9″	12″	15″	18″	21″	24″
2	1.4	.89	.48	.32	.25	.20	.16	.14	.11
3	2.3	1.7	.93	.64	.48	.39	.32	.27	.25
4	3.4	2.5	1.5	1.1	.82	.66	.55	.48	.41
5	4.3	3.4	2.2	1.6	1.2	.98	.82	.71	.61
6	5.5	4.5	3.0	2.2	1.7	1.4	1.1	.98	.86
7	6.4	5.5	3.9	2.7	2.2	1.8	1.5	1.3	1.2
8	7.5	6.6	4.8	3.4	2.7	2.3	1.9	1.7	1.5
9	8.4	7.5	5.7	4.3	3.4	2.7	2.3	2.1	1.8
10	9.5	8.6	6.6	5.0	4.1	3.4	2.7	2.5	2.2
11	10.	9.8	7.7	5.9	4.8	4.1	3.4	3.0	2.5
12	12.	11.	8.6	6.8	5.7	4.5	4.1	3.4	3.0

Fig. 3.



N \ L	3″	6″	9″	12″	15″	18″	21″	24″
4	1.8	.96	.64	.50	.40	.32	.28	.22
6	3.4	1.9	1.3	.96	.78	.64	.54	.50
8	5.0	3.0	2.2	1.6	1.3	1.1	.96	.82
10	6.8	4.4	3.2	2.4	2.0	1.6	1.4	1.2
12	9.0	6.0	4.4	3.4	2.8	2.2	2.0	1.7
14	11.0	7.8	5.4	4.4	3.6	3.0	2.6	2.4
16	13.2	9.6	6.8	5.4	4.6	3.8	3.4	3.0
18	15.0	11.4	8.6	6.8	5.4	4.6	4.2	3.6
20	17.2	13.2	10.0	8.2	6.8	5.4	5.0	4.4
22	19.6	15.4	11.8	9.6	8.2	6.8	6.0	5.0
24	22.0	17.2	13.6	11.4	9.0	8.2	6.8	6.0



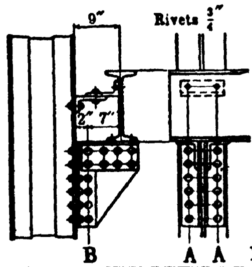
Rivets $\frac{3}{4}$″ A. B. Co. Specifications.

L = 12″
D = 7½″
N = 12
C = 4.0
S = 5,300#
W = 21,200# = (C × S)

$\frac{21,200\#}{6}$ = 3,500# on one rivet.

$\frac{3}{8}$″ plate is O. K.

Fig. 1.



Rivets $\frac{3}{4}$″ A. B. Co. Specifications.

Rivets "A"
L = 9″
C = 4.4
S = 5,300#
W = 23,300# = (C × S)

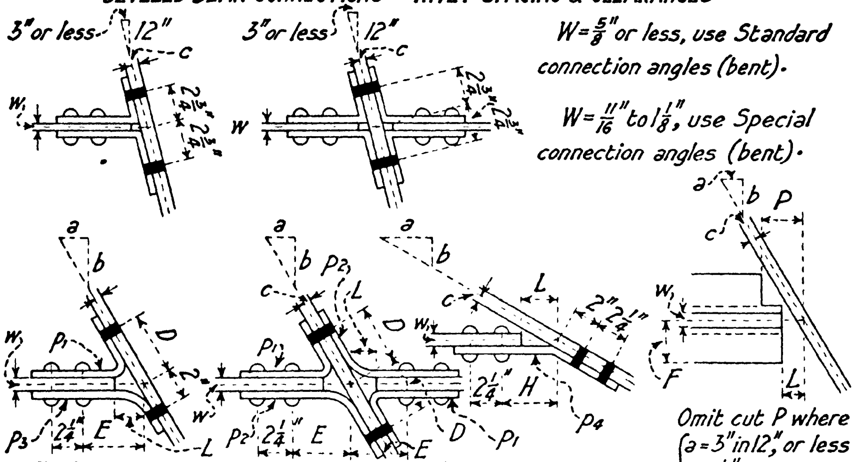
Rivets "B"
L = 7″
C = 2.7
W = 23,300#
S = $\frac{23,300\#}{2.7}$ = 8,600#

Fig. 2. use ½″ plate.

TABLE 119.

Standard Beveled Beam Connections.
American Bridge Company.

BEVELED BEAM CONNECTIONS – RIVET SPACING & CLEARANCES



W = 5/8" or less, use Standard connection angles (bent).

W = 11/16" to 1 1/8", use Special connection angles (bent).

Omit cut P where a = 3" in 12", or less; c = 1/4" or less; F = 3" or less.

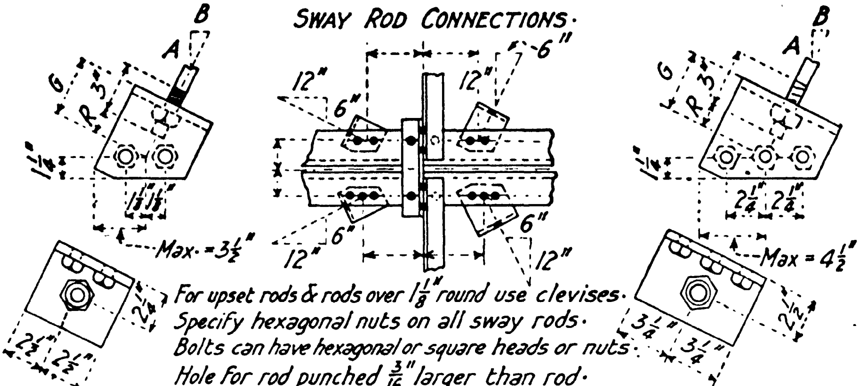
For large duplication modify these details where necessary to permit machine riveting. Table covers plates up to 1/2" thick.

a	b	Max. c	Max. W	D	E	H	Length of Bent Plates p₁	p₂	p₃	p₄	L	P F = up to 3"	P F = 3" to 4"
1"	12"	9/16"	1 1/8"	2 3/4"	See notes above.						1"	1 1/2"	1 1/2"
2	12	9/16	1 1/8	2 3/4							1 1/4	1 3/4	1 3/4
3	12	9/16	1 1/8	2 3/4							1 1/4	2	2 1/4
4	12	9/16	1 1/8	3 1/2	3 1/2"	2 3/4"	10"	11 1/2"	10"	12"	1 1/4	2 1/4	2 1/2
5	12	9/16	1 1/8	4	4	3	11	12 1/2	10 1/2	12	1 1/2	2 1/2	3
6	12	9/16	1 1/8	4 1/2	4 1/2	3	12	13 1/2	11	12	1 1/2	2 3/4	3 1/4
7	12	9/16	1 1/8	5	5	3 1/4	12 1/2	14 1/2	11 1/2	12	1 3/4	3	3 1/2
8	12	9/16	1 1/8	5 1/2	5 1/2	3 1/4	13	15 1/2	12	12	1 3/4	3 1/4	4
9	12	9/16	1 1/8			3 1/4				12	1 3/4	3 1/2	4 1/2
10	12	9/16	1 1/8			3 1/2				12 1/2	2	4	5
11	12	9/16	1 1/8			3 1/2				12 1/2	2	4 1/2	5 1/2
12	12	9/16	1 1/8			3 3/4				12 1/2	2 1/4	4 1/2	5 1/2
12	11	1/4	1/2			2 3/4				12	1 1/4	4 1/2	5 1/2
12	10	1/4	1/2			3				12	1 1/2	5	6
12	9	1/4	1/2			3				12	1 1/2	5 1/2	6 1/2
12	8	1/4	1/2			3 1/4				12	1 3/4	6	7 1/2
12	7	1/4	1/2			3 1/2				12 1/2	2	6 1/2	8 1/2
12	6	1/4	1/2			3 3/4				12 1/2	2 1/4	7 1/2	10
12	5	1/4	1/2			4				13	2 1/2	9	11 1/2
12	4	1/4	1/2			4 3/4				13 1/2	3 1/4	11	14

TABLE 120.

Standard Sway Rod and Lateral Connections.
American Bridge Company.

SWAY ROD CONNECTIONS.



For upset rods & rods over $1\frac{1}{8}''$ round use clevises.
Specify hexagonal nuts on all sway rods.
Bolts can have hexagonal or square heads or nuts.
Hole for rod punched $\frac{3}{16}''$ larger than rod.

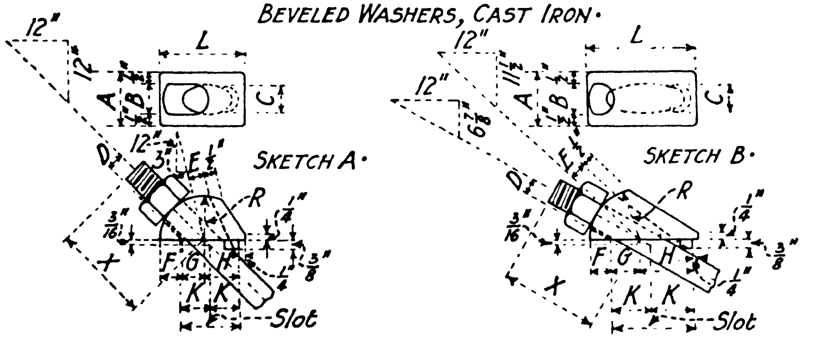
Rod $\frac{3}{4}''$ round (not upset), Bolts $\frac{3}{4}''$ or $\frac{5}{8}''$ round.
Rod $\frac{7}{8}''$ round (not upset), Bolts $\frac{3}{4}''$ or $\frac{5}{8}''$ round.

A	B	Size of Angle	G	R
12"	6" to 12"	$6'' \times 4'' \times \frac{1}{2}''$, 5" long	$3\frac{3}{4}''$	$1\frac{1}{2}''$
6" to 12"	12"	$6'' \times 4'' \times \frac{1}{2}''$, 5 long	$3\frac{3}{4}''$	$1\frac{1}{2}''$

Rod 1" round (not upset), Bolts $\frac{3}{4}''$ or $\frac{5}{8}''$ round.
Rod $1\frac{1}{8}''$ round (not upset), Bolts $\frac{3}{4}''$ round.

A	B	Size of Angle	G	R
12"	6" to 12"	$6'' \times 4'' \times \frac{5}{8}''$, $6\frac{1}{2}''$ long	$3\frac{1}{4}''$	$\frac{3}{4}''$
6" to 12"	12"	$8'' \times 4'' \times \frac{5}{8}''$, $6\frac{1}{2}$ long	$4\frac{1}{2}''$	2"

BEVELED WASHERS, CAST IRON.



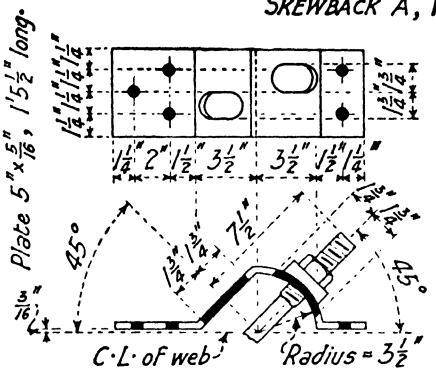
Sketch	Round Rod	Upset	A	B	C	D	E	F	G	H	L	R	X	K	Size of Slot in Plate	Weight Pounds
A	$\frac{7}{8}''$ 1"	None	$2\frac{1}{8}''$	$1\frac{1}{8}''$	1"	$\frac{9}{16}''$	$\frac{9}{16}''$	$\frac{7}{8}''$	$\frac{7}{8}''$	$1\frac{3}{8}''$	$3\frac{3}{8}''$	$1\frac{3}{4}''$	4"	$1\frac{1}{8}''$	$1\frac{1}{8}'' \times 2\frac{1}{4}''$	1·8
A	1 $1\frac{1}{8}$	$1\frac{3}{8}''$ $1\frac{1}{2}$	$2\frac{5}{8}$	$1\frac{5}{8}$	$1\frac{1}{2}$	$\frac{13}{16}$	$\frac{13}{16}$	$\frac{7}{8}$	$1\frac{1}{8}$	$1\frac{7}{8}$	$4\frac{1}{8}$	2	5	$1\frac{5}{8}$	$1\frac{3}{4} \times 3\frac{1}{4}$	2·6
B	$\frac{7}{8}$ 1	None	$2\frac{1}{8}$	$1\frac{1}{8}$	1	$\frac{9}{16}$	$\frac{9}{16}$	$\frac{3}{4}$	$1\frac{1}{4}$	$2\frac{1}{4}$	$4\frac{1}{2}$	2	$4\frac{1}{2}$	$1\frac{3}{4}$	$1\frac{1}{8}'' \times 3\frac{1}{2}''$	2·3
B	1 $1\frac{1}{8}$	$1\frac{3}{8}$ $1\frac{1}{2}$	$2\frac{5}{8}$	$1\frac{5}{8}$	$1\frac{1}{2}$	$\frac{13}{16}$	$\frac{13}{16}$	$\frac{3}{4}$	$1\frac{7}{8}$	$3\frac{1}{8}$	6	$2\frac{5}{8}$	6	$2\frac{5}{8}$	$1\frac{3}{4} \times 5\frac{1}{4}$	3·8

For rods above $1\frac{1}{8}''$ diam. use clevis connections.

TABLE 121.

Standard Lateral Connections for Highway Bridges.
American Bridge Company.

Skewback "A", Weight 6·8 lbs·



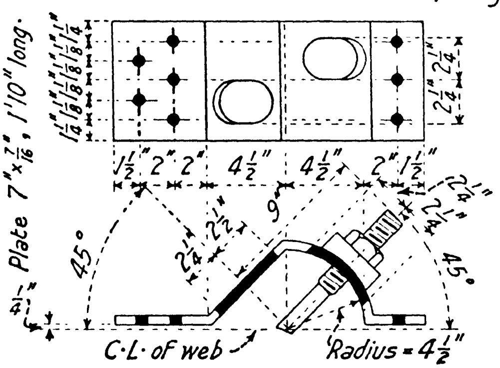
Skewback "A" for rods up to $1\frac{1}{4}$" round or $1\frac{1}{8}$" square (upset to $1\frac{5}{8}$" round);

For upsets $1\frac{1}{8}$" diam· or less, angle of rod may vary from 32° ($7\frac{1}{2}$" in 12") to 60° (12" in $6\frac{15}{16}$")·

For upsets greater than $1\frac{1}{8}$" diam· up to $1\frac{5}{8}$" diam·, angle of rod may vary from $41\frac{1}{2}$° ($10\frac{5}{8}$" in 12") to 60° (12" in $6\frac{15}{16}$")·

Standard slot in beam $3\frac{1}{2}$" × 6".

Skewback "B", Weight 17 lbs·



Skewback "B" for rods $1\frac{1}{4}$" round or $1\frac{1}{8}$" square (upset to $1\frac{5}{8}$" round);

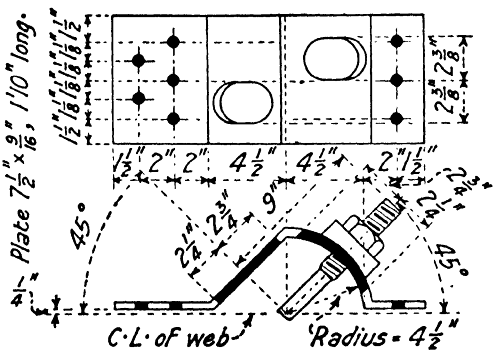
up to $\begin{cases} 1\frac{1}{2}\text{" round (upset to } 1\frac{7}{8}\text{" round) or} \\ 1\frac{3}{8}\text{" square (upset to 2" round·)} \end{cases}$

For upsets $1\frac{5}{8}$" diam· or less, angle of rod may vary from $33\frac{2}{3}$° (8" in 12") to 60° (12" in $6\frac{15}{16}$")·

For upsets greater than $1\frac{5}{8}$" diam· up to 2" diam·, angle of rod may vary from $38\frac{2}{3}$° ($9\frac{19}{32}$" in 12") to 60° (12" in $6\frac{15}{16}$")·

Standard slot in beam $4\frac{1}{4}$" × $6\frac{1}{2}$"·

Skewback "C", Weight 23 lbs·



Skewback "C" for rods $1\frac{9}{16}$" round or $1\frac{7}{16}$" square (upset to 2" round);

up to $\begin{cases} 1\frac{3}{4}\text{" round (upset to } 2\frac{1}{8}\text{" round) or} \\ 1\frac{1}{2}\text{" square (upset to } 2\frac{1}{4}\text{" round)} \end{cases}$

Angle of rod may vary from $40\frac{1}{2}$° ($10\frac{1}{4}$" in 12") to $64\frac{1}{3}$° (12" in $5\frac{3}{4}$") for all rods·

Standard slot in beam $4\frac{3}{4}$" × $6\frac{1}{2}$"

Where upset end of rod is greater than $2\frac{1}{8}$" diam·, hole in washer will be drilled to fit upset·

TABLE 122.

STANDARD LATERAL CONNECTIONS AND STUB ENDS.
AMERICAN BRIDGE COMPANY.

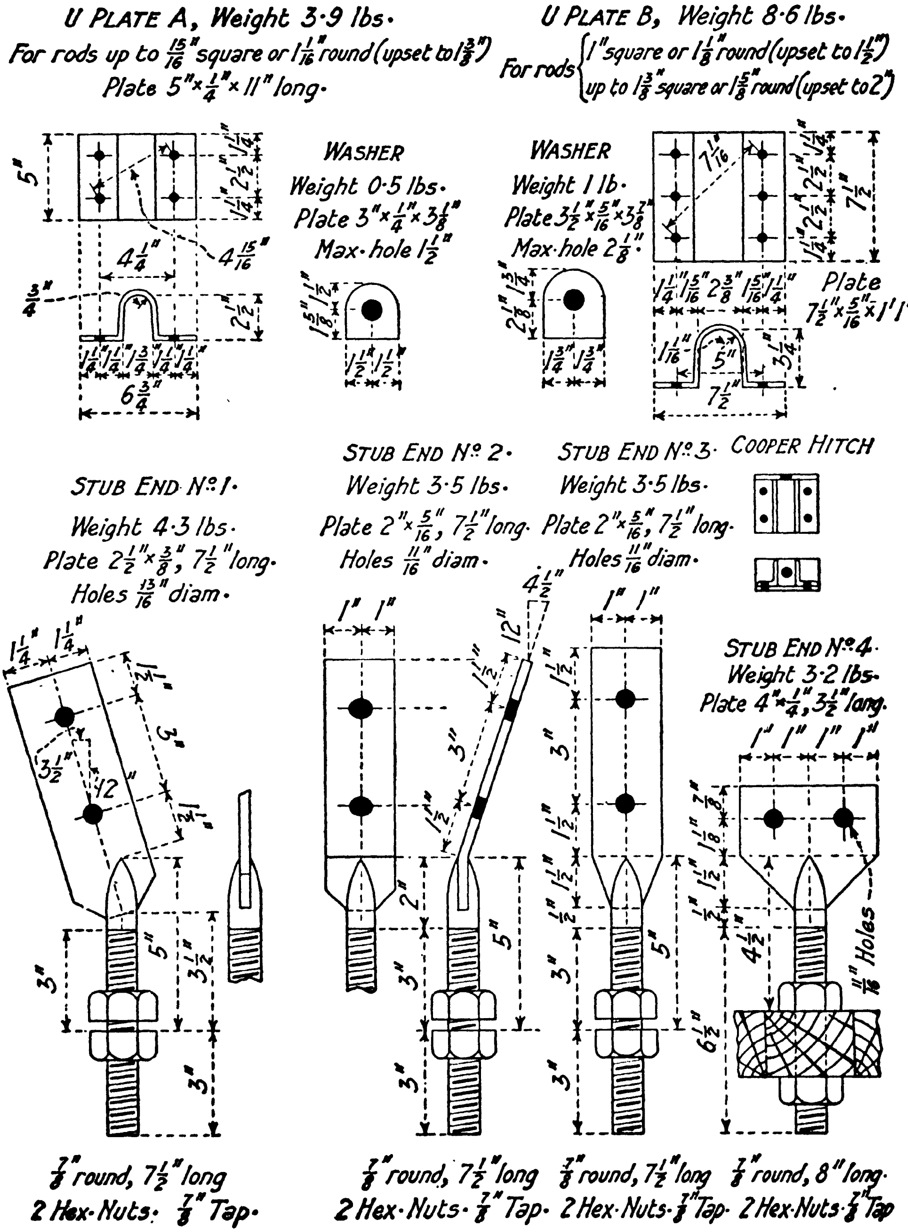
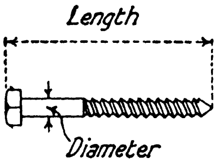


TABLE 123.

STANDARD LAG SCREWS, HOOK BOLTS AND WASHERS.
AERRICAN BRIDGE COMPANY.

LAG SCREWS

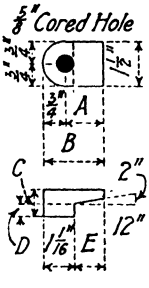


Diam	Min. Length	Max. Length	No. Thread per inch
$\frac{5}{16}$"	$1\frac{1}{2}$"	6"	
$\frac{3}{8}$	$1\frac{1}{2}$	6	
$\frac{7}{16}$	$1\frac{1}{2}$	8	
$\frac{1}{2}$	$1\frac{1}{2}$	10	
$\frac{9}{16}$	2	12	
$\frac{5}{8}$	2	12	5
$\frac{3}{4}$	$2\frac{1}{2}$	12	4
$\frac{7}{8}$	3	12	3
1	$3\frac{1}{2}$	12	
$1\frac{1}{8}$	5	12	
$1\frac{1}{4}$	6	12	
$1\frac{1}{2}$	8	12	

Length of Lag Screw & Head	
Length of Screw	Length of Head
$1\frac{1}{2}$"	1"
2	$1\frac{1}{4}$
$2\frac{1}{2}$	$1\frac{1}{2}$
3	$1\frac{3}{4}$
$3\frac{1}{2}$	2
4	$2\frac{1}{4}$
$4\frac{1}{2}$	$2\frac{1}{2}$
5	$2\frac{3}{4}$
$5\frac{1}{2}$	3
6	$3\frac{1}{4}$
7	$3\frac{3}{4}$
8	$4\frac{1}{4}$
9	[illegible]
10	5
11	5
12	5

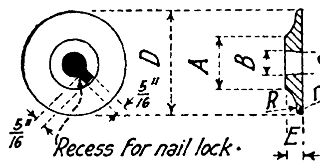
Heads are the same as for square head bolts.
Threaded portion is not tapered except at point.

BEAM CLAMP



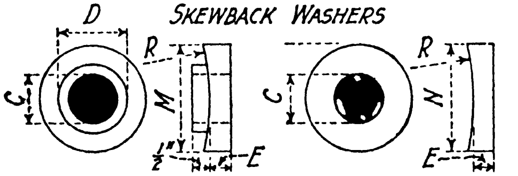
Size Beam	Dimensions of Clamp A	B	C	D	E	Weight in lbs.
18"	$1\frac{1}{2}$"	$2\frac{1}{4}$"	$\frac{7}{8}$"	$\frac{15}{32}$"	$1\frac{3}{16}$"	0.4
15	$1\frac{1}{2}$	$2\frac{1}{4}$	$\frac{7}{8}$	$\frac{13}{32}$	$1\frac{3}{16}$	0.4
12	$1\frac{1}{2}$	$2\frac{1}{4}$	$\frac{7}{8}$	$\frac{11}{32}$	$1\frac{3}{16}$	0.4
9&10	$1\frac{1}{2}$	$2\frac{1}{4}$	$\frac{3}{4}$	$\frac{5}{16}$	$1\frac{3}{16}$	0.4
7&8	$1\frac{1}{4}$	2	$\frac{3}{4}$	$\frac{1}{4}$	$\frac{15}{16}$	0.4
5&6	$1\frac{1}{4}$	2	$\frac{5}{8}$	$\frac{7}{32}$	$\frac{15}{16}$	0.3

OGEE WASHERS



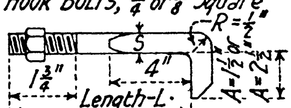
Size Bolt	Dimensions of Washer A	B	C	D	E	R	r	Weight in Pounds
$\frac{5}{8}$"	$1\frac{3}{8}$"	$\frac{3}{4}$"	$\frac{11}{16}$"	$2\frac{3}{4}$"	$\frac{1}{2}$"	$\frac{5}{8}$"	$\frac{3}{16}$"	0.4
$\frac{3}{4}$	$1\frac{5}{8}$	$\frac{7}{8}$	$\frac{13}{16}$	$3\frac{1}{4}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{32}$	0.7
$\frac{7}{8}$	$1\frac{7}{8}$	$1\frac{1}{16}$	$\frac{31}{32}$	$3\frac{3}{4}$	$\frac{3}{4}$	$\frac{7}{8}$	$\frac{1}{4}$	1.0

SKEWBACK WASHERS



Used With	Dimensions of Washers M	N	C	D	E	R	Weight in Pounds
Skewback "A"	$2\frac{3}{4}$"		$1\frac{1}{4}$"	$1\frac{5}{8}$"	$\frac{3}{4}$"	$3\frac{13}{16}$"	1.2
		$3\frac{1}{4}$"	$1\frac{1}{2}$		$\frac{7}{8}$	$3\frac{13}{16}$	1.8
		$3\frac{3}{4}$	$1\frac{3}{4}$		1	$3\frac{13}{16}$	2.5
Skewback "B"	$3\frac{3}{4}$"		$1\frac{3}{4}$"	$2\frac{1}{8}$"	1"	$4\frac{15}{16}$"	2.7
		4"	2		1	$4\frac{15}{16}$	3.0
		$4\frac{1}{2}$	$2\frac{1}{4}$		1	$4\frac{15}{16}$	3.9

HOOK BOLTS, $\frac{3}{4}$" or $\frac{7}{8}$" Square,



In billing Hook Bolts give dimensions A, S & L; all other dimensions are standard. Unless otherwise specified, "S" will be made $\frac{3}{4}$". Hex. nuts furnished.

CAST IRON CUP WASHERS

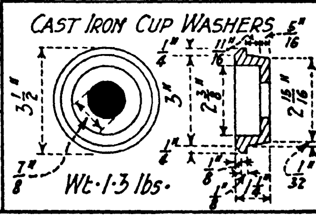


TABLE 124.

Weights of Washers and Track Bolts.

Weights of Lag Screws.

Pounds per Hundred. (Kent's Pocket-book.)

Diam.	Length, Under Head, in Inches.															
In.	1½	1¾	2	2¼	2½	3	3½	4	4½	5	5½	6	7	8	9	10
3/8	6.88	7.50	8.25	9.25	9.62	10.82	11.50	13.31	14.82	16.50	17.37	18.82	......	......	......	......
7/16	......	11.75	12.62	12.88	13.28	16.62	18.18	18.88	19.50	21.25	23.56	25.31	......	......	......	......
1/2	......	16.88	17.18	18.07	19.18	22.00	24.00	26.82	28.25	30.37	33.88	35.37	38.94	44.37	......	......
5/8	......	......	......	......	......	34.07	35.88	39.25	42.62	47.75	51.62	55.12	61.88	68.75	77.00	90.00
3/4	......	......	......	......	......	......	......	64.00	67.88	71.37	79.37	86.62	92.75	97.50	108.75	124.75

For American Bridge Company's Standard Lag Screws see Table 123.

Wrought Iron or Steel Plate Round Washers.

Diam.	Hole.	Thickness B.W.G.	Bolt.	Number in 200 Lb.	Diam.	Hole.	Thickness B.W.G.	Bolt.	Number in 200 Lb.	Diam.	Hole.	Thickness B.W.G.	Bolt.	Number in 200 Lb.
In.	In.	No.	In.		In.	In.	No.	In.		In.	In.	No.	In.	
9/16	1/4	18	3/16	85200	1½	5/8	12	9/16	4600	3	1¼	9	1¼	900
3/4	5/16	16	1/4	34800	1¾	11/16	10	5/8	2600	3¼	1⅜	8	1⅜	600
7/8	3/8	16	5/16	26200	2	13/16	10	3/4	2200	3½	1½	8	1½	570
1	7/16	14	3/8	14400	2¼	15/16	9	7/8	1600	3¾	1⅝	8	1⅝	460
1¼	1/2	14	7/16	8400	2½	1 1/16	9	1	1200	4	1¾	8	1¾	432
1⅜	9/16	12	1/2	5800	2¾	1¼	9	1⅛	888	4¼	1⅞	8	1⅞	366

Standard Cast, O G Washers.

Diam. of Bolt.	Bottom Diam.	Top Diam.	Hole.	Thickness.	Weight.	Diam. of Bolt.	Bottom Diam.	Top Diam.	Hole.	Thickness.	Weight.
In.	In.	In.	In.	In.	Lb.	In.	In.	In.	In.	In.	Lb.
1/2	2⅝	1¾	9/16	5/8	1/2	1⅛	4¾	2¾	1 3/16	1⅛	3
5/8	3	1⅞	11/16	3/4	3/4	1¼	6	3	1 5/16	1¼	5¾
3/4	3¼	2⅛	13/16	7/8	1¼	1½	6¼	3¼	1⅝	1½	6
7/8	3¾	2½	15/16	7/8	1½	1¾	7¼	3¾	1⅞	1¾	9½
1	4	2¾	1 1/16	1⅛	2½	2	8¼	4¼	2⅛	2	17¼

Track Bolts.

With United States Standard Hexagon Nuts.

Wt. of Rail per Yd.	Bolts.	Nuts.	No. in Keg. 200 Lb.	Kegs per Mile.	Wt. of Rail per Yd.	Bolts.	Nuts.	No. in Keg. 200 Lb.	Kegs per Mile.	Wt. of Rail per Yd.	Bolts.	Nuts.	No. in Keg. 200 Lb.	Kegs per Mile.
Lb.	In.	In.			Lb.	In.	In.			Lb.	In	In.		
45 to 85	¾x4½	1¼	230	6.3	45 to 85	¾x3	1¼	283	5.1	20 to 30	½x3	7/8	715	2
	¾x4	1¼	240	6.0	30 to 40	⅝x3½	1 1/16	375	4.0		½x2½	7/8	760	2
	¾x3¾	1¼	254	5.7		⅝x3	1 1/16	410	3.7		½x2¼	7/8	800	2
	¾x3½	1¼	260	5.5		⅝x2¾	1 1/16	435	3.3		½x2	7/8	820	2
	¾x3¼	1¼	266	5.4		⅝x2½	1 1/16	465	3.1					

TABLE 125.

Weights of Steel Wire Nails and Spikes.

American Steel and Wire Co.

Standard Steel Wire Nails and Spikes.

Sizes, Lengths and Approximate Number per Pound.

Size.	Length. In.	Common Nails and Brads.	Flooring Brads.	Finishing.	Casing and Smooth or Barbed Box.	Slating.	Shingle.	Barbed Car. Heavy.	Barbed Car. Light.	Hinge. Heavy.	Hinge. Light.	Fence.	Clinch.	Fine.	Lining.	Barbed Roofing.	Barrel.	Wire Spikes.	Length. In.	Size.
	[illegible]																1615		[illegible]	
	[illegible]														2077	714	1346		[illegible]	
	[illegible]														1781	469	906		[illegible]	
2d Ex. Fine	1													1560					1	2d Ex. Fine
2d	1	876		1351	1010	411							710	1351	1558	411	775		1	2d
3d Ex. Fine	1⅛													1015		365	700		1⅛	3d Ex. Fine
3d	1¼	568		807	635	225	568						429	778		251	568		1¼	3d
	1⅜															230	400		1⅜	
4d	1½	316		584	473	187	274	165	274	50	82		274	473		176	357		1½	4d
5d	1¾	271		500	406	142	235	118	142			142	235			151			1¾	5d
6d	2	181	157	309	236	103	204	103	124	38	62	124	157			103			2	6d
7d	2¼	161	139	238	210		139	76	92			92	139						2¼	7d
8d	2½	106	99	189	145		125	69	82	30	50	82	99						2½	8d
9d	2¾	96	90	172	132		114	54	62			62	90						2¾	9d
10d	3	69	69	121	94		83	50	57	12	25	50	69					41	3	10d
12d	3¼	63	54	113	88			42	50	11	23	40	62					38	3¼	12d
16d	3½	49	43	90	71			35	43	10	22	30	49					30	3½	16d
20d	4	31	31	62	52			26	31	9	19	23	37					23	4	20d
30d	4½	24			46			24	28									17	4½	30d
40d	5	18			35			18	21									13	5	40d
50d	5½	14						15	17									10	5½	50d
60d	6	11						13	15									8	6	60d
[illegible]/16 Diam.	7																	7	7	[illegible]/16 Diam.
"	8																	6	8	"
"	9																	5	9	"
"	10																	4	10	"
"	12																	3	12	"

Miscellaneous Steel Wire Nails.

Approximate Number per Pound.

Washburn & Moen Gauge.	Diameter in Inches.	Length in Inches. ½	⅝	¾	1	1¼	1½	1¾	2	2½	3	3½	4	4½	5	6	7	8	9	10
000	.362					28	23	20	17	14	12	10	9	8	7	6	5	4½	4	3½
00	.331					33	27	23	20	16	14	12	10	9	8	7	6	5	4½	4
0	.307					38	32	27	24	19	16	14	12	10	9	8	7	6	5	4¼
1	.283				57	45	38	32	28	23	19	16	14	13	11	10	8	7	6	5½
2	.263				65	52	44	37	32	26	22	19	16	14	13	11	9	8	7	6½
3	.244			100	76	60	50	43	38	30	25	22	19	17	15	13	11	10	8	7½
4	.225			120	90	72	60	51	45	36	30	26	23	20	18	15	13	11	10	9
5	.207	211	169	141	106	85	71	60	53	42	35	30	26	24	21	18	15			
6	.192	247	197	164	123	99	82	71	62	50	41	35	31	28	25	21	18			
7	.177	299	239	200	149	120	100	85	75	60	50	43	37	33	30	25				
8	.162	345	275	229	172	137	115	98	86	69	57	49	43	39	35	29				
9	.148	414	331	276	207	165	138	118	103	82	69	59	52	46	41					
10	.135	496	397	333	248	198	165	142	124	99	83	71	62	55	50					
11	.120	628	502	418	314	251	209	179	157	125	105	90	79	70						
12	.105	822	658	548	411	329	274	235	204	164	137	117	103							
13	.092	1072	857	714	536	429	357	306	268	214	178	153								
14	.080	1420	1136	947	710	568	473	406	350	284	236									
15	.072	1752	1402	1168	876	701	584	500	438	350										
16	.063	2280	1828	1523	1143	913	761	653	571											
17	.054	3116	2495	2077	1558	1246	1038	800	779											
18	.047	4138	3310	2758	2069	1655	1379	1182												
19	.041	5334	4267	3556	2667	2133	1778													
20	.035	7500	6000	5000	3750	3000														
21	.032	8888	7111	5926	4444															
22	.028	11428	9143	7618																

W. & M. Gauge.	11	12
000	3¼	3
00	3¾	3½
0	4¼	4
1	5	4½
2	6	5½

These approximate numbers are an average only, and the figures given may be varied either way, by changes in the dimensions of heads or points. Brads and no-head nails will have more to the pound than table shows, and large or thick-headed nails will have less.

TABLE 126.

WEIGHTS OF NAILS AND SPIKES.

FROM CAMBRIA STEEL.

CUT STEEL NAILS AND SPIKES.

Sizes, Lengths and Approximate Number per Pound.

Sizes.	Length. Inches.	Common.	Clinch.	Finishing.	Casing and Box.	Fencing.	Spikes.	Barrel.	Light Barrel.	Slating.	Sizes.	Length. Inches.	Flat Grip. Fine.	Edge Grip. Fine.
	5/8							750				3/4	1462	
	3/4							600				7/8	1300	
	7/8							500			2d	1	1100	960
2d	1	740	400	1100				450		340	3d	1 1/8	800	750
	1 1/8							310	400		4d	1 1/4	650	600
3d	1 1/4	460	260	880				280	304	280				
	1 3/8							210			To-bacco.		Brads.	Shingle.
4d	1 1/2	280	180	530	420			190	224	220				
5d	1 3/4	210	125	350	300	100				180	130			
6d	2	160	100	300	210	80					97		120	
7d	2 1/4	120	80	210	180	60					85		94	
8d	2 1/2	88	68	168	130	52					68		74	90
9d	2 3/4	73	52	130	107	38					58		62	72
10d	3	60	48	104	88	26					48		50	60
12d	3 1/4	46	40	96	70	20							40	
16d	3 1/2	33	34	86	52	18	17						27	
20d	4	23	24	76	38	16	14							
25d	4 1/4	20												
30d	4 1/2	16 1/2			30		11							
40d	5	12			26		9							
50d	5 1/2	10			20		7 1/2							
60d	6	8			16		6							
	6 1/2						5 1/2							
	7						5							

SQUARE BOAT SPIKES.

Approximate Number in a Keg of 200 Pounds.

Length of Spike—Inches.

Size.	3	4	5	6	7	8	9	10	Size.	6	7	8	9	10	11	12	14	16
1/4"	3000	2375	2050	1825					7/16"	600	590	510	400	360	320	230		
5/16"	1660	1360	1230	1175	990	880			1/2"	450	375	335	300	275	260	240		
3/8"	1320	1140	940	800	650	600	525	475	5/8"			260	240	220	205	190	175	160

RAILROAD SPIKES.

Size Under Head.	Average Number per Keg of 200 Lb.	Spikes per Mile of Single Track. Ties 2 Ft. c. to c., 4 Spikes per Tie.		Rail Used. Weight per Yard.	Size Under Head.	Average Number per Keg of 200 Lb.	Spikes per Mile of Single Track. Ties 2 Ft. c to c., 4 Spikes per Tie.		Rail Used. Weight per Yard.
Inches.		Pounds.	Kegs.	Pounds.	Inches.		Pounds.	Kegs.	Pounds.
5 1/2 × 5/8	300	7040	35 1/2	75 to 100	4 1/2 × 7/16	680	3110	15 1/2	20 to 30
5 1/2 × 9/16	375	5870	29 1/2	45 " 75	4 × 7/16	720	2910	14 1/2	20 " 30
5 × 9/16	400	5170	26	40 " 56	3 1/2 × 7/16	900	2350	11	16 " 25
5 × 1/2	450	4660	23 1/2	35 " 40	4 × 3/8	1000	2090	10 1/2	16 " 25
4 1/2 × 1/2	530	3960	20	30 " 35	3 1/2 × 3/8	1190	1780	9	16 " 20
4 × 1/2	600	3520	17 3/4	25 " 35	3 × 3/8	1240	1710	8 1/2	26 " 20

TABLE 127.

Pipe—Black and Galvanized.

NATIONAL TUBE COMPANY STANDARD.

Standard Pipe.

Size, In.	Diameters, Inches.		Thickness, Inches.	Weight per Foot, Pounds.		Threads per Inch.	Couplings.		
	External.	Internal.		Plain Ends.	Threads and Couplings.		Diameter, Inches.	Length, Inches.	Weight, Pounds.
$\frac{1}{8}$	.405	.269	.068	.244	.245	27	.562	$\frac{7}{8}$	.029
$\frac{1}{4}$	.540	.364	.088	.424	.425	18	.685	1	.043
$\frac{3}{8}$	.675	.493	.091	.567	.568	18	.848	$1\frac{1}{8}$	.070
$\frac{1}{2}$	.840	.622	.109	.850	.852	14	1.024	$1\frac{3}{8}$	.116
$\frac{3}{4}$	1.050	.824	.113	1.130	1.134	14	1.281	$1\frac{5}{8}$	.209
1	1.315	1.049	.133	1.678	1.684	$11\frac{1}{2}$	1.576	$1\frac{7}{8}$	.343
$1\frac{1}{4}$	1.660	1.380	.140	2.272	2.281	$11\frac{1}{2}$	1.950	$2\frac{1}{8}$	.535
$1\frac{1}{2}$	1.900	1.610	.145	2.717	2.731	$11\frac{1}{2}$	2.218	$2\frac{3}{8}$	.743
2	2.375	2.067	.154	3.652	3.678	$11\frac{1}{2}$	2.760	$2\frac{5}{8}$	1.208
$2\frac{1}{2}$	2.875	2.469	.203	5.793	5.819	8	3.276	$2\frac{7}{8}$	1.720
3	3.500	3.068	.216	7.575	7.616	8	3.948	$3\frac{1}{8}$	2.498
$3\frac{1}{2}$	4.000	3.548	.226	9.109	9.202	8	4.591	$3\frac{5}{8}$	4.241
4	4.500	4.026	.237	10.790	10.889	8	5.091	$3\frac{5}{8}$	4.741
$4\frac{1}{2}$	5.000	4.506	.247	12.538	12.642	8	5.591	$3\frac{5}{8}$	5.241
5	5.563	5.047	.258	14.617	14.810	8	6.296	$4\frac{1}{8}$	8.091
6	6.625	6.065	.280	18.974	19.185	8	7.358	$4\frac{1}{8}$	9.554
7	7.625	7.023	.301	23.544	23.769	8	8.358	$4\frac{1}{8}$	10.932
8	8.625	8.071	.277	24.696	25.000	8	9.358	$4\frac{5}{8}$	13.905
8	8.625	7.981	.322	28.554	28.809	8	9.358	$4\frac{5}{8}$	13.905
9	9.625	8.941	.342	33.907	34.188	8	10.358	$5\frac{1}{8}$	17.236
10	10.750	10.192	.279	31.201	32.000	8	11.721	$6\frac{1}{8}$	29.877
10	10.750	10.136	.307	34.240	35.000	8	11.721	$6\frac{1}{8}$	29.877
10	10.750	10.020	.365	40.483	41.132	8	11.721	$6\frac{1}{8}$	29.877
11	11.750	11.000	.375	45.557	46.247	8	12.721	$6\frac{1}{8}$	32.550
12	12.750	12.090	.330	43.773	45.000	8	13.958	$6\frac{1}{8}$	43.098
12	12.750	12.000	.375	49.562	50.706	8	13.958	$6\frac{1}{8}$	43.098
13	14.000	13.250	.375	54.568	55.824	8	15.208	$6\frac{1}{8}$	47.152
14	15.000	14.250	.375	58.573	60.375	8	16.446	$6\frac{1}{8}$	59.493
15	16.000	15.250	.375	62.579	64.500	8	17.446	$6\frac{1}{8}$	63.294

The permissible variation in weight is 5 per cent above and 5 per cent below.
Furnished with threads and couplings and in random lengths unless otherwise ordered.
Taper of threads is $\frac{3}{4}''$ diameter per foot length for all sizes.
The weight per foot of pipe with threads and couplings is based on a length of 20 feet including the coupling, but shipping lengths of small sizes will usually average less than 20 feet.
All weights and dimensions are nominal. On sizes made in more than one weight, weight desired must be specified.

TABLE 127.—*Continued.*

PIPE—BLACK AND GALVANIZED—Concluded.

NATIONAL TUBE COMPANY STANDARD.

EXTRA STRONG PIPE.

Size, In.	Diameters, Inches. External.	Diameters, Inches. Internal.	Thickness, Inches.	Weight per Foot, Pounds. Plain Ends.
1/8	.405	.215	.095	.314
1/4	.540	.302	.119	.535
3/8	.675	.423	.126	.738
1/2	.840	.546	.147	1.087
3/4	1.050	.742	.154	1.473
1	1.315	.957	.179	2.171
1¼	1.660	1.278	.191	2.996
1½	1.900	1.500	.200	3.631
2	2.375	1.939	.218	5.022
2½	2.875	2.323	.276	7.661
3	3.500	2.900	.300	10.252
3½	4.000	3.364	.318	12.505
4	4.500	3.826	.337	14.983
4½	5.000	4.290	.355	17.611
5	5.563	4.813	.375	20.778
6	6.625	5.761	.432	28.573
7	7.625	6.625	.500	38.048
8	8.625	7.625	.500	43.388
9	9.625	8.625	.500	48.728
10	10.750	9.750	.500	54.735
11	11.750	10.750	.500	60.075
12	12.750	11.750	.500	65.415
13	14.000	13.000	.500	72.091
14	15.000	14.000	.500	77.431
15	16.000	15.000	.500	82.771

DOUBLE EXTRA STRONG PIPE.

Size, In.	Diameters, Inches. External.	Diameters, Inches. Internal.	Thickness, Inches.	Weight per Foot, Pounds. Plain Ends.
1/2	.840	.252	.294	1.714
3/4	1.050	.434	.308	2.440
1	1.315	.599	.358	3.659
1¼	1.660	.896	.382	5.214
1½	1.900	1.100	.400	6.408
2	2.375	1.503	.436	9.029
2½	2.875	1.771	.552	13.695
3	3.500	2.300	.600	18.583
3½	4.000	2.728	.636	22.850
4	4.500	3.152	.674	27.541
4½	5.000	3.580	.710	32.530
5	5.563	4.063	.750	38.552
6	6.625	4.897	.864	53.160
7	7.625	5.875	.875	63.079
8	8.625	6.875	.875	72.424

Furnished with plain ends and in random lengths unless otherwise ordered.

Permissible variation in weight, for extra strong pipe, 5 per cent above and 5 per cent below.

For double extra strong pipe, 10 per cent above and 10 per cent below.

All weights and dimensions are nominal.

LARGE O. D. PIPE.

Size, In.	Weight per Foot, Pounds. Thickness, Inches. 1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	7/8	1
14	36.713	45.682	54.568	63.371	72.091	80.726	89.279	106.134	122.654	138.842
15	39.383	49.020	58.573	68.044	77.431	86.734	95.954	114.144	132.000	149.522
16	42.053	52.357	62.579	72.716	82.771	92.742	102.629	122.154	141.345	160.202
17	44.723	55.695	66.584	77.389	88.111	98.749	109.304	130.164	150.690	170.882
18	47.393	59.032	70.589	82.061	93.451	104.757	115.979	138.174	160.035	181.562
20	....	65.708	78.599	91.407	104.131	116.772	129.330	154.194	178.725	202.923
21	....	69.045	82.604	96.079	109.471	122.780	136.005	162.204	....	....
22	....	72.383	86.609	100.752	114.811	128.787	142.680	170.215	....	....
24	....	....	94.619	110.097	125.491	140.802	156.030	186.235	....	....
26	....	....	102.629	119.442	136.172	152.818	169.380	202.255	....	....
28	....	....	....	128.787	146.852	164.833	182.730	218.275	....	....
30	....	....	....	138.132	157.532	176.848	196.081	234.296	....	....

Furnished with plain ends and in random lengths, unless otherwise ordered.
All weights and dimensions are nominal.

TABLE 128.

Standard Gages. Comparative Table.

Carnegie Steel Co.

Gage Number.	Thickness in Decimals of an Inch.						
	Birmingham Wire (B. W. G.) also known as Stubs Iron Wire.	United States Standard for Sheet and Plate Iron and Steel.	American Wire or Browne & Sharpe.	American Steel & Wire Co. formerly Washburn & Moen.	Trenton Iron Company.	British Imperial Standard Wire (S. W. G.).	Standard Birmingham Sheet and Hoop (B. G.).
0000000	......	.500	......	.4900	......	.500	......
000000	......	.46875	.580000	.4615	......	.464	......
00000	.500	.4375	.516500	.4305	.450	.432	......
0000	.454	.40625	.460000	.3938	.400	.400	......
000	.425	.375	.409642	.3625	.360	.372	.5000
00	.380	.34375	.364796	.3310	.330	.348	.4452
0	.340	.3125	.324861	.3065	.305	.324	.3964
1	.300	.28125	.289297	.2830	.285	.300	.3532
2	.284	.265625	.257627	.2625	.265	.276	.3147
3	.259	.25	.229423	.2437	.245	.252	.2804
4	.238	.234375	.204307	.2253	.225	.232	.2500
5	.220	.21875	.181940	.2070	.205	.212	.2225
6	.203	.203125	.162023	.1920	.190	.192	.1981
7	.180	.1875	.144285	.1770	.175	.176	.1764
8	.165	.171875	.128490	.1620	.160	.160	.1570
9	.148	.15625	.114423	.1483	.145	.144	.1398
10	.134	.140625	.101897	.1350	.130	.128	.1250
11	.120	.125	.090742	.1205	.1175	.116	.1113
12	.109	.109375	.080808	.1055	.105	.104	.0991
13	.095	.09375	.071962	.0915	.0925	.092	.0882
14	.083	.078125	.064084	.0800	.0806	.080	.0785
15	.072	.0703125	.057068	.0720	.070	.072	.0699
16	.065	.0625	.050821	.0625	.061	.064	.0625
17	.058	.05625	.045257	.0540	.0525	.056	.0556
18	.049	.05	.040303	.0475	.045	.048	.0495
19	.042	.04375	.035890	.0410	.040	.040	.0440
20	.035	.0375	.031961	.0348	.035	.036	.0392
21	.032	.034375	.028462	.03175	.031	.032	.0349
22	.028	.03125	.025346	.0286	.028	.028	.03125
23	.025	.028125	.022572	.0258	.025	.024	.02782
24	.022	.025	.020101	.0230	.0225	.022	.02476
25	.020	.021875	.017900	.0204	.020	.020	.02204
26	.018	.01875	.015941	.0181	.018	.018	.01961
27	.016	.0171875	.014195	.0173	.017	.0164	.01745
28	.014	.015625	.012641	.0162	.016	.0148	.015625
29	.013	.0140625	.011257	.0150	.015	.0136	.0139
30	.012	.0125	.010025	.0140	.014	.0124	.0123
31	.010	.0109375	.008928	.0132	.013	.0116	.0110
32	.009	.01015625	.007950	.0128	.012	.0108	.0098
33	.008	.009375	.007080	.0118	.011	.0100	.0087
34	.007	.00859375	.006305	.0104	.010	.0092	.0077
35	.005	.0078125	.005615	.0095	.0095	.0084	.0069
36	.004	.00703125	.005000	.0090	.009	.0076	.0061
37	......	.006640625	.004453	.0085	.0085	.0068	.0054
38	......	.00625	.003965	.0080	.008	.0060	.0048
39	......	......	.003531	.0075	.0075	.0052	......
40	......	......	.003144	.0070	.007	.0048	......

Unless otherwise specified, all orders in gages will be executed to Birmingham Wire Gage.

TABLE 129.

Standard Gages and Weights of Sheet Steel.

Carnegie Steel Co.

UNITED STATES STANDARD GAGE
For
Sheet and Plate Steel.

Gage Number.	Thickness in Fractions of an Inch.	Thickness in Decimals of an Inch.	Weight per Square Foot, in Pounds, Steel.	Gage Number.	Thickness in Fractions of an Inch.	Thickness in Decimals of an Inch.	Weight per Square Foot, in Pounds, Steel.
0000000	$\frac{1}{2}$	.5	20.4	17	$\frac{9}{160}$	.05625	2.295
000000	$\frac{15}{32}$	.46875	19.125	18	$\frac{1}{20}$	.05	2.04
00000	$\frac{7}{16}$	.4375	17.85	19	$\frac{7}{160}$	.04375	1.785
0000	$\frac{13}{32}$	.40625	16.575	20	$\frac{3}{80}$	.0375	1.53
000	$\frac{3}{8}$	.375	15.3	21	$\frac{11}{320}$	.034375	1.4025
00	$\frac{11}{32}$	.34375	14.025	22	$\frac{1}{32}$	.03125	1.275
0	$\frac{5}{16}$	.3125	12.75	23	$\frac{9}{320}$	.028125	1.1475
1	$\frac{9}{32}$	.28125	11.475	24	$\frac{1}{40}$	.025	1.02
2	$\frac{17}{64}$	.265625	10.8375	25	$\frac{7}{320}$	.021875	.8925
3	$\frac{1}{4}$	.25	10.2	26	$\frac{3}{160}$	.01875	.765
4	$\frac{15}{64}$	.234375	9.5625	27	$\frac{11}{640}$	.0171875	.70125
5	$\frac{7}{32}$	.21875	8.925	28	$\frac{1}{64}$	.015625	.6375
6	$\frac{13}{64}$	.203125	8.2875	29	$\frac{9}{640}$	.0140625	.57375
7	$\frac{3}{16}$	.1875	7.65	30	$\frac{1}{80}$	.0125	.51
8	$\frac{11}{64}$	.171875	7.0125	31	$\frac{7}{640}$	.0109375	.44625
9	$\frac{5}{32}$	.15625	6.375	32	$\frac{13}{1280}$	.01015625	.414375
10	$\frac{9}{64}$	.140625	5.7375	33	$\frac{3}{320}$	.009375	.3825
11	$\frac{1}{8}$	.125	5.1	34	$\frac{11}{1280}$	.00859375	.350625
12	$\frac{7}{64}$	.109375	4.4625	35	$\frac{5}{640}$	.0078125	.31875
13	$\frac{3}{32}$	.09375	3.825	36	$\frac{9}{1280}$	.00703125	.286875
14	$\frac{5}{64}$	.078125	3.1875	37	$\frac{17}{2560}$	.006640625	.2709375
15	$\frac{9}{128}$	.0703125	2.86875	38	$\frac{1}{160}$	.00625	.255
16	$\frac{1}{16}$	.0625	2.55				

BIRMINGHAM WIRE GAGE.
Equivalents in Inches.
Corresponding Weights of Flat Rolled Steel.

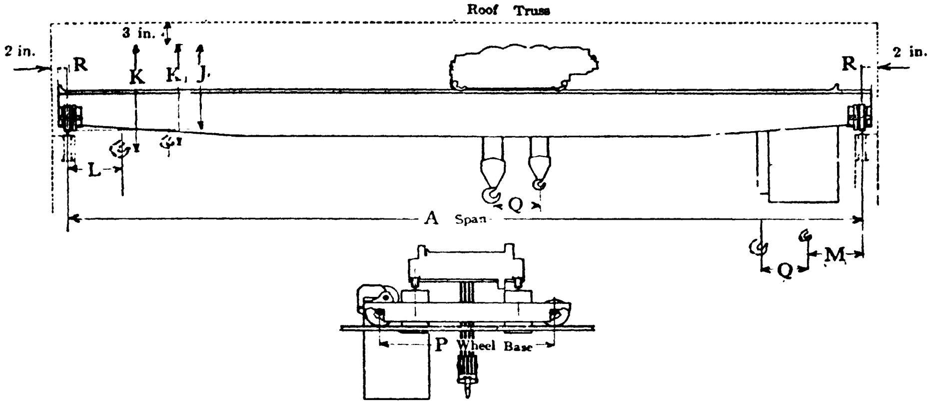
Gage Number.	Thickness, Inches.	Pounds per Square Foot.	Gage Number.	Thickness, Inches.	Pounds per Square Foot.
0000	.454	18.5232	17	.058	2.3664
000	.425	17.34	18	.049	1.9992
......	....	....	19	.042	1.7136
00	.380	15.504	20	.035	1.428
0	.340	13.872	21	.032	1.3056
1	.300	12.24	22	.028	1.1424
2	.284	11.5872	23	.025	1.02
3	.259	10.5672	24	.022	0.8976
......	....	....	25	.020	0.816
4	.238	9.7104	26	.018	0.7344
5	.220	8.976	27	.016	0.6528
6	.203	8.2824	28	.014	0.5712
7	.180	7.344	29	.013	0.5304
8	.165	6.732	30	.012	0.4896
9	.148	6.0384	31	.010	0.408
10	.134	5.4672	32	.009	0.3672
11	.120	4.896	33	.008	0.3264
12	.109	4.4472	34	.007	0.2856
13	.095	3.876	35	.005	0.2040
14	.083	3.3864	36	.004	0.1632
15	.072	2.9376			
16	.065	2.651			

TABLE 130.

Clearance Dimensions and Wheel Loads, Electric Cranes.

Shaw, Crane Works, Muskegon, Mich.

(McClintic-Marshall Construction Company Standards).



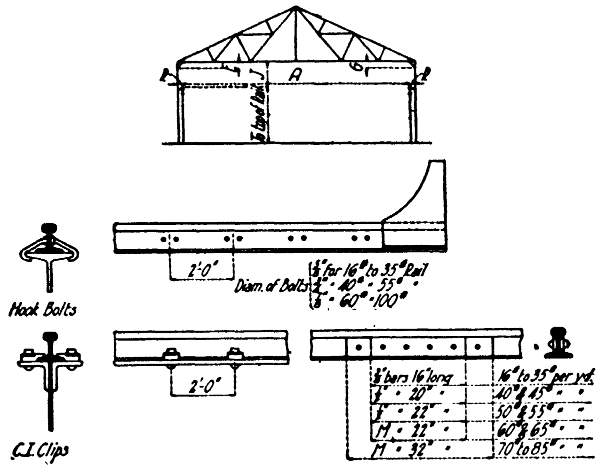
Capacity.	A	R	J	K	K1	L	M	Q	P	Wt. of Runway Rail.	Max. Wheel Load.
	Ft.	In.	Ft.-In.	Ft.-In.	Ft.-In.	Ft.-In.	Ft.-In.	Ft.-In.	Ft.-In.	Lb.	Lb.
5 Ton	40	8	4-10	5- 4	......	3- 4	MAIN HOOK 2- 6	......	9-10	40	13,600
	50	8	5- 1	5- 4	......	3- 4	2- 6	......	10- 4	40	15,000
	60	8	5- 1	5- 4	......	3- 4	2- 6	......	10- 6	40	16,600
	70	8	5- 2	5- 5	......	3- 4	2- 6	......	11- 8	40	18,500
	80	8	5- 2	5- 5	......	3- 4	2- 6	......	13- 4	40	20,000
	90	8	5-11	5- 5	......	3- 4	2- 6	......	15- 0	60	23,000
10 Ton	40	8	5- 3	5-10	......	3- 6	MAIN HOOK 2- 6	......	10- 6	40	20,200
	50	8	5- 7	5-10	......	3- 6	2- 6	......	11- 0	60	22,300
	60	8	5- 7	5-10	......	3- 6	2- 6	......	11- 2	60	23,600
	70	8	6- 1	5-10	......	3- 6	2- 6	......	11- 8	60	25,400
	80	8	6- 1	5-10	......	3- 6	2- 6	......	13- 4	60	28,800
	90	8	6- 1	5-10	......	3- 6	2- 6	......	15- 0	60	32,600
15 Ton	40	8	5-10	8- 7	......	3- 8	MAIN HOOK 2-11	......	10- 8	60	27,300
	50	8	5-10	8- 7	......	3- 8	2-11	......	11- 0	60	29,400
	60	8	5-10	8- 7	......	3- 8	2-11	......	11- 2	60	30,700
	70	10	6- 6	8- 7	......	3- 8	2-11	......	11- 8	70	33,500
	80	10	6- 6	8- 7	......	3- 8	2-11	......	13- 4	70	36,700
	90	10	6- 6	8- 7	......	3- 8	2-11	......	15- 0	70	38,600
20 Ton with 5 Ton Aux.	40	10	6- 7	8- 7	7-0	3- 8	2- 6	3- 0	11- 4	70	35,400
	50	10	6- 7	8- 7	7-0	3- 8	2- 6	3- 0	11- 8	70	38,000
	60	10	6- 7	8- 7	7-0	3- 8	2- 6	3- 0	11-10	70	39,600
	70	10	7- 0	8- 7	7-0	3- 8	2- 6	3- 0	12- 2	70	41,700
	80	10	7- 0	8- 7	7-0	3- 8	2- 6	3- 0	13- 4	70	44,700
	90	12	7- 3	8- 7	7-0	3- 8	2- 6	3- 0	15- 0	90	48,000
30 Ton with 5 Ton Aux.	50	12	7- 2	9- 9	7-3	3-10	2- 8	3- 2	12-10	90	51,700
	60	12	7- 2	9- 9	7-3	3-10	2- 8	3- 2	13- 0	90	54,700
	70	12	7- 7	9- 9	7-3	3-10	2- 8	3- 2	13- 4	90	57,600
	80	12	7- 7	9- 9	7-3	3-10	2- 8	3- 2	13- 8	90	62,100
	90	12	7- 7	9- 9	7-3	3-10	2- 8	3- 2	15- 0	90	65,400
40 Ton with 10 Ton Aux.	50	13	8- 0	11- 1	8-5	4- 4	2-11	3- 6	12-10	90	65,300
	60	13	8- 2	11- 1	8-5	4- 4	2-11	3- 6	13- 0	100	70,400
	70	13	8- 2	11- 1	8-5	4- 4	2-11	3- 6	13- 4	100	74,300
	80	13	8- 2	11- 1	8-5	4- 4	2-11	3- 6	13- 8	100	79,400
	90	13	8- 2	11- 1	8-5	4- 4	2-11	3- 6	15- 0	100	84,800
50 Ton with 10 Ton Aux.	50	13	8- 9	12- 3	8-7	4- 6	3- 0	3-11	13- 6	100	82,000
	60	13	8- 9	12- 3	8-7	4- 6	3- 0	3-11	13- 8	100	87,300
	70	13	8- 9	12- 3	8-7	4- 6	3- 0	3-11	14- 0	100	91,200

Greater Capacities and Spans Require 8 Wheels

TABLE 131.

Crane Girder Specifications.

McClintic-Marshall Construction Co.



Weight of Rail per Yard.	Weight of Rail Splices per Pair with Bolts.	Weight of Rail Clamp.	Weight of Hook Bolts.	Crane Stop.		Area of Rail.	Height and Width of Base of Rail.	Web of Rail.	Width of Head of Rail.
				Plates.	Cast Iron.				
Lb.	Lb.	Lb.	Lb.	Lb.	Lb.	Sq. In.	In.	In.	In.
16	5	....	.5	....	....	1.6	$2\frac{1}{4}$	$\frac{15}{64}$	$1\frac{1}{4}$
20	5	....	.5	....	....	2.0	$2\frac{1}{2}$	$\frac{17}{64}$	$1\frac{3}{8}$
25	5	2.7	.5	....	....	2.5	$2\frac{3}{4}$	$\frac{19}{64}$	$1\frac{1}{2}$
30	5	2.7	.5	....	....	3.0	3	$\frac{21}{64}$	$1\frac{5}{8}$
35	5	2.7	.5	....	....	3.4	$3\frac{1}{4}$	$\frac{23}{64}$	$1\frac{3}{4}$
40	13	2.7	.9	56	35	3.9	$3\frac{1}{2}$	$\frac{25}{64}$	$1\frac{7}{8}$
45	13	3.2	.9	56	35	4.4	$3\frac{11}{16}$	$\frac{27}{64}$	2
50	15	"	.9	56	35	4.9	$3\frac{7}{8}$	$\frac{7}{16}$	$2\frac{1}{8}$
55	15	"	1.3	57	35	5.4	$4\frac{1}{16}$	$\frac{15}{32}$	$2\frac{1}{4}$
60	14	"	1.3	57	35	5.9	$4\frac{1}{4}$	$\frac{31}{64}$	$2\frac{3}{8}$
65	14	"	1.3	57	35	6.4	$4\frac{7}{16}$	$\frac{1}{2}$	$2\frac{13}{32}$
70	22	"	1.4	74	50	6.9	$4\frac{5}{8}$	$\frac{33}{64}$	$2\frac{7}{16}$
75	22	"	1.4	74	50	7.4	$4\frac{13}{16}$	$\frac{17}{32}$	$2\frac{15}{32}$
80	22	"	1.5	74	50	7.8	5	$\frac{35}{64}$	$2\frac{1}{2}$
85	23	"	1.5	74	50	8.3	$5\frac{3}{16}$	$\frac{9}{16}$	$2\frac{9}{16}$
90	79.2	"	1.5	75	50	8.8	$5\frac{3}{8}$	$\frac{9}{16}$	$2\frac{5}{8}$
95	86.2	"	1.5	75	50	9.3	$5\frac{9}{16}$	$\frac{9}{16}$	$2\frac{11}{16}$
100	92.4	"	1.5	75	50	9.8	$5\frac{3}{4}$	$\frac{9}{16}$	$2\frac{3}{4}$

Crane Rails: Crane Rails are attached to the girder by means of clips or hook bolts, the latter being used chiefly for I-Beams, the flange being too narrow for a clip, and has the advantage of saving punching in the top flange. Clips and hook bolts provide for adjusting slight inaccuracies in the alignment of the rails. Rail Splices should consist of a flat bar fish plate or a rolled fish plate as angle splices are apt to interfere with the flange of the crane wheels. Provide our standard crane stop at the end of the rail.

Dimensions: In preparing design indicate clearly distances A, R, J, E, G and distances of floor line to top of rail. These dimensions should be submitted to owners with design, but before ordering or manufacturing any material for the work the owner's approval should be obtained for same.

TABLE 132.

Typical Hand Cranes.

McClintic-Marshall Construction Co.

Capacity.	Span.	Wheel Base.	Max. Wheel Load.	Vertical Clearance.	Side Clearance.	Wt. of Rails.		Capacity.	Span.	Wheel Base.	Max. Wheel Load.	Vertical Clearance.	Side Clearance.	Wt. of Rails.	
						I-Beams.	Plate Girders.							I-Beams.	Plate Girders.
Tons.	Ft.	Ft.	Lb.	Ft.	In.	Lb. per Yd.		Tons.	Ft.	Ft.	Lb.	Ft.	In.	Lb. per Yd.	
2	30	4	3100	4	7	30	30	10	30	7	13000	5	10	40	40
2	50	5	4000	4	7	30	30	10	50	8	14400	5	10	40	40
4	30	4	5400	4½	8	30	30	12	30	7	20700	5½	10	45	45
4	50	5	6500	4½	8	30	30	12	50	8	22300	5½	10	45	45
6	30	6	8000	5	9	30	35	14	30	7	26000	5½	10	50	50
6	50	7	9200	5	9	30	35	14	50	8	28000	5½	10	50	50
8	30	6	10500	5	10	35	40	16	30	7	32300	6	12	50	55
8	50	7	11800	5	10	35	40	16	50	8	35000	6	12	50	55

TABLE 133.
Details of a Steel Stair

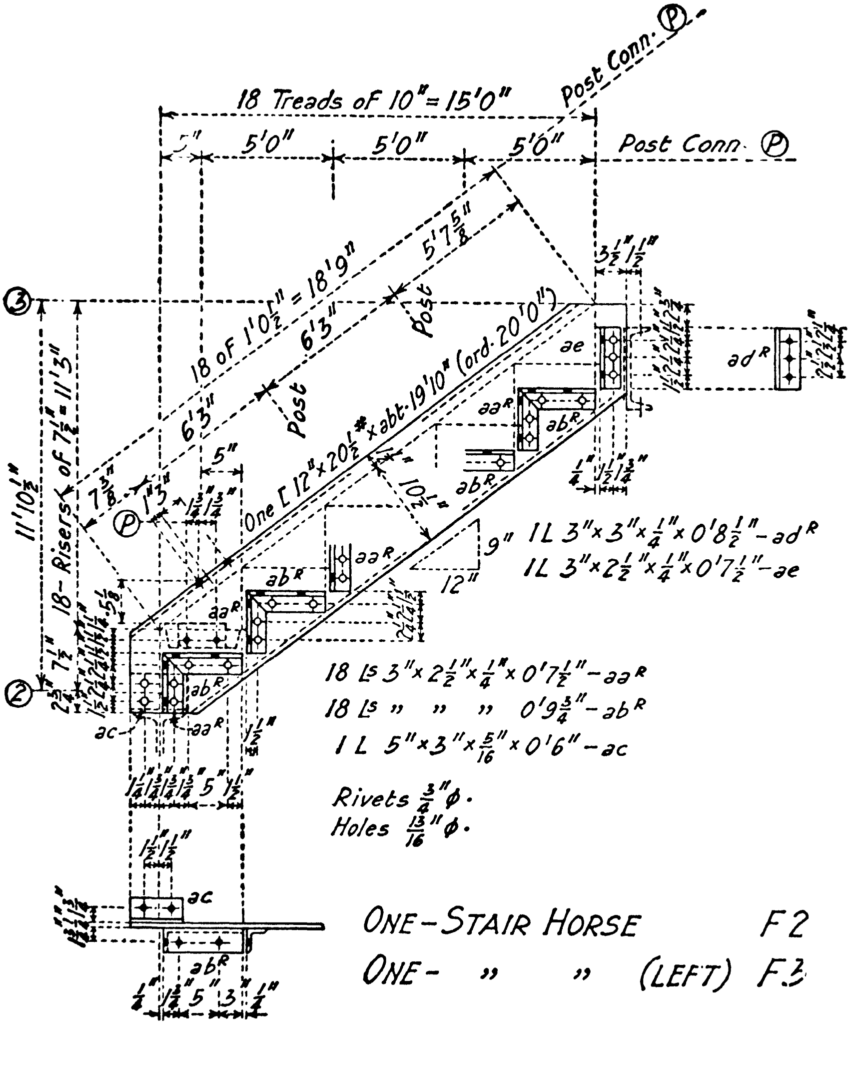
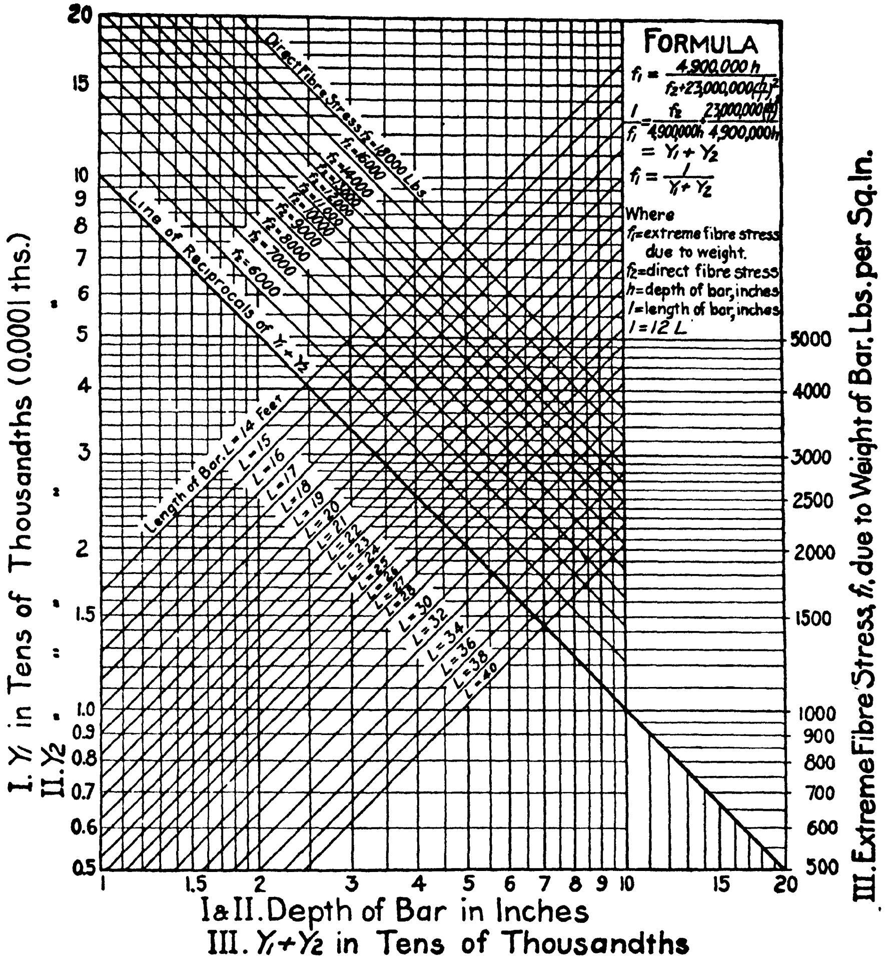


TABLE 134.

Diagram for Stress in Eye-bars Due to Weight.



Problem.—Required stress due to weight of a 4 in. x 1 in. eye-bar, 20 ft. long, which has a direct tension of 56,000 lb.

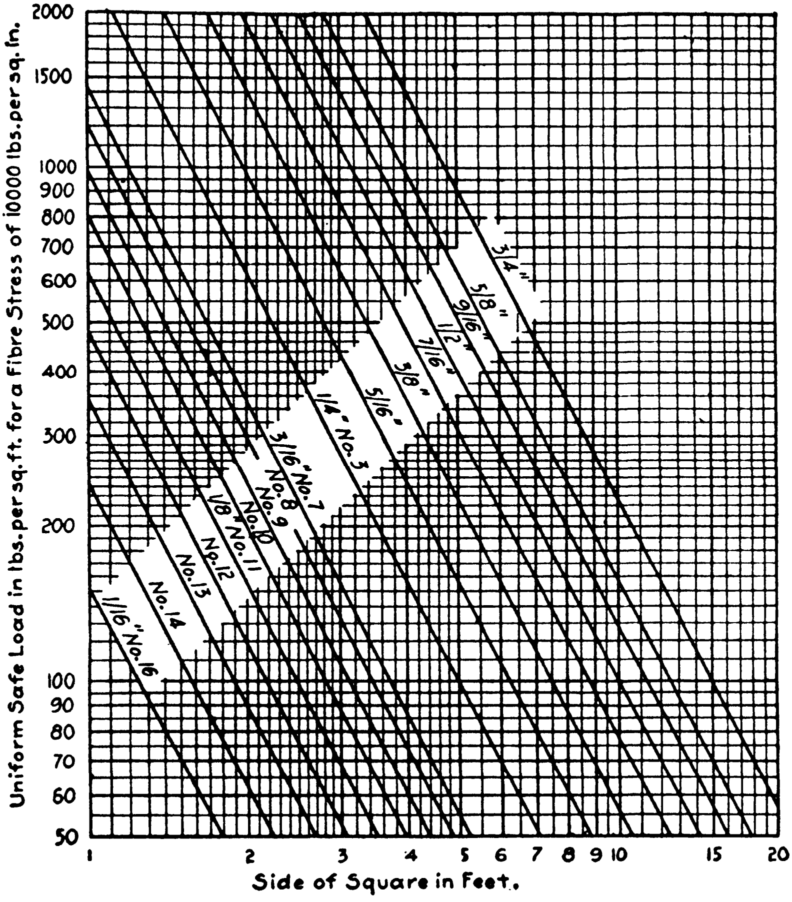
Then, h = 4 in.; L = 20 ft., and f_2 = 14,000 lb. per sq. in. The stress due to weight, f_1, is found from the diagram as follows: On the bottom of the diagram, find h = 4 in.; follow up the vertical line to its intersection with inclined line marked, L = 20 ft., then follow the horizontal line passing through the point of intersection out to the left margin and find, y_2 = 3.3 tens of thousandths; then follow vertical line, h = 4 in., up to its intersection with inclined line marked, f_2 = 14,000, and then follow the horizontal line passing through the point of intersection to left margin and find, y_1 = 7.2 tens of thousandths. Now $y_1 + y_2$ = 7.2 + 3.3 = 10.5. Find $y_1 + y_2$ = 10.5 on lower edge of diagram, follow vertical line to its intersection with line marked "Line of Reciprocals" and find on right margin, f_1 = 950 lb. sq. in.

For a bar inclined at an angle θ with a vertical line multiply the fiber stress calculated for a horizontal bar as above, of the same length, and multiply the fiber stress thus obtained by sin θ. For example if the bar above is inclined at an angle of 45 degrees with the vertical; the fiber stress due to weight is, f_1 = 950 x sin θ = 950 x 0.707 = 672 lb.

Every intersection of the inclined f_2 and L lines has for its abscissa a value of h, which will have a maximum fiber stress, f_1, for the given values of f_2 and L. For example for L = 30 ft.; f_2 = 12,000 lb., we find h = 8.3 in., and f_1 = 1,700 lb. A deeper or shallower bar will give a smaller value of f_1.

TABLE 135.

DIAGRAM FOR STRESSES IN SQUARE PLATES.



Safe Loads on Square Plates.—The safe loads on square plates for a fiber stress of 10,000 pounds per square inch may be obtained from the diagram. As an example, required the safe load for a ½-in. plate 3 feet square. Begin at 3 on the bottom of the diagram, follow upward to the line marked ½-in. plate, from the intersection follow to the left edge and find 280 lb. per sq. ft. For any other fiber stress multiply the safe load found from the diagram by the ratio of the fiber stresses. To use the diagram for a rectangular plate take a square plate having the same area.

For formulas for strength of plates, see page 313, Chapter VIII.

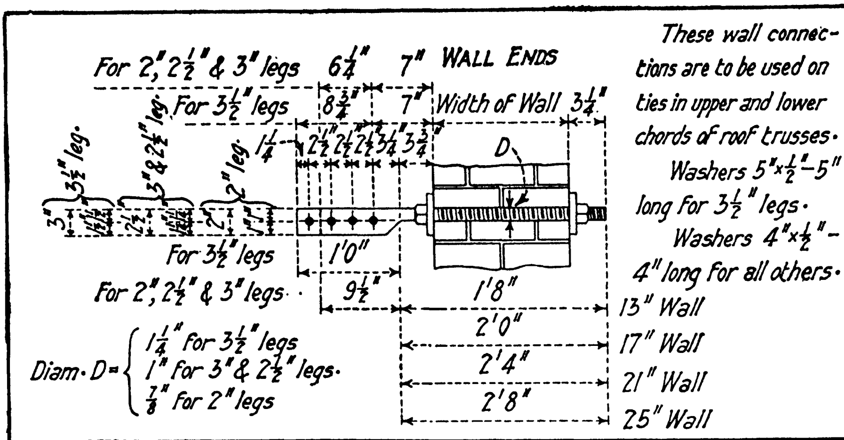
TABLE 136.

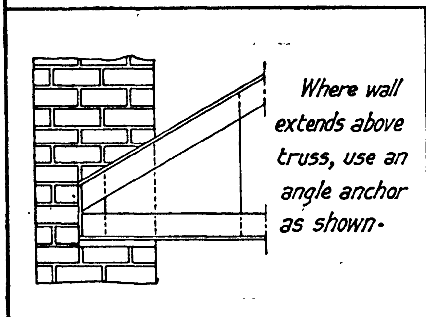
APPROXIMATE RADII OF GYRATION OF VARIOUS STRUCTURAL SECTIONS.			
$r_A = 0.29d$, $r_B = 0.29b$	$r_A = 0.41d$, $r_B = 0.41b$; Mean d & b	$r_A = 0.25d$, $r_B = 0.25d$	$r_A = 0.35d$, $r_B = 0.35d$; d = Mean diam.
$r_A = 0.4d$, $r_B = 0.2b$	$r_A = 0.38d$, $r_B = 0.28b$	$r_A = 0.31d$, $r_B = 0.29b$	$r_A = 0.30d$, $r_B = 0.30b$
$r_A = 0.29d$, $r_B = 0.23b$	$r_A = 0.30d$, $r_B = 0.22b$	$r_A = 0.31d$, $r_B = 0.21b$	$r_A = 0.30d$, $r_B = 0.30d$, $r_C = 0.19d$
$r_A = 0.32d$, $r_B = 0.49b$	$r_A = 0.38d$, $r_B = 0.60b$	$r_A = 0.38d$, $r_B = 0.45b$	$r_A = 0.45d$, $r_B = 0.31b$
$r_A = 0.31d$, $r_B = 0.54b$	$r_A = 0.31d$, $r_B = 0.45b$	$r_A = 0.29d$, $r_B = 0.5b$	$r_A = 0.29d$, $r_B = 0.45b$
$r_A = 0.31d$, $r_B = 0.48b$	$r_A = 0.4d$, $r_B = 0.24b$	$r_A = 0.41d$, $r_B = 0.22b$	$r_A = 0.38d$, $r_B = 0.21b$
$r_A = 0.4d$, $r_B = 0.23b$	$r_A = 0.39d$, $r_B = 0.21b$	$r_A = 0.45d$, $r_B = 0.24b$	$r_A = 0.36d$, $r_B = 0.32b$
$r_A = 0.36d$, $r_B = 0.45b$	$r_A = 0.45d$, $r_B = 0.3b$	$r_A = 0.42d$, $r_B = 0.42b$	$r_A = 0.45d$, $r_B = 0.3b$
$r_A = 0.31d$, $r_B = 0.18b$	$r_A = 0.25d$, $r_B = 0.21b$	$r_A = 0.32d$, $r_B = 0.32b$	$r_A = 0.25d$, $r_B = 0.37b$
$r_A = 0.25d$, $r_B = 0.32b$	$r_A = 0.39d$, $r_B = 0.32b$	$r_A = 0.4d$, $r_B = 0.34b$	$r_A = 0.4d$, $r_B = 0.33b$

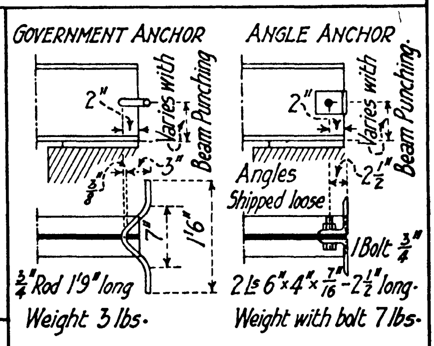
TABLE 137.

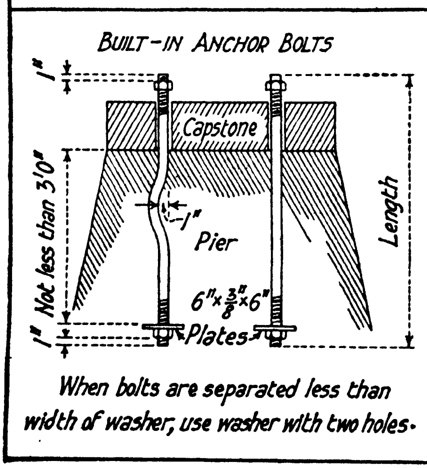
DETAILS OF ANCHORS AND ANCHOR BOLTS.

AMERICAN BRIDGE COMPANY.

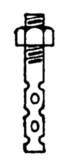








SWEDGE BOLTS



Diameter Inches	Length Ft. & In.	Weight with nut
¾	0–9	1.3#
⅞	1–0	2.3#
1	1–0	3.1#
1¼	1–3	6.1#

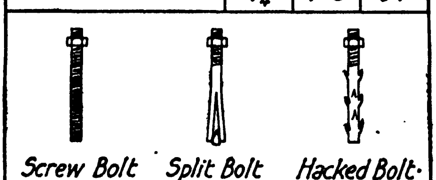
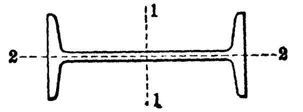


TABLE 151

Properties of Bethlehem I Beams



Depth of Beam	Weight per Foot	Area	Thickness of Web	Width of Flange	Increase of Web and Flange for Each Pound Increase in Weight	Moment of Inertia, Axis 1–1, I_1	Moment of Inertia, Axis 2–2, I_2	Radius of Gyration, Axis 1–1, r_1	Radius of Gyration, Axis 2–2, r_2	Section Modulus, Axis 1–1, S_1	Maximum Safe Shear on Web	Maximum Bending Moment @ 16,000 Lbs. per Sq. In., M_1	Add for Each Pound Increase in Weight, m	Spacing Center to Center to Produce Equal Radii of Gyration About Each Axis
In.	Lb.	$In.^2$	In.	In.	In.	$In.^4$	$In.^4$	In.	In.	$In.^3$	Lb.	Ft.-Lb.	Ft.-Lb.	In.
30	121	35.30	.540	10.500	.010	5 239.6	165.0	12.18	2.16	349.3	103 800	465 740	1 960	23.98
28	106	30.88	.500	10.000	.011	4 014.1	131.5	11.40	2.06	286.7	89 000	382 300	1 830	22.43
26	91	26.49	.460	9.500	.011	2 977.2	101.2	10.60	1.95	229.0	75 300	305 350	1 700	20.84
24	84.5	24.80	.460	9.250	.012	2 381.9	91.1	9.80	1.92	198.5	75 100	264 660	1 570	19.22
	83	24.59	.520	9.130	.012	2 240.9	78.0	9.55	1.78	186.7	93 100	248 980	1 570	18.76
	73.5	21.47	.390	9.000	.012	2 091.0	74.4	9.87	1.86	174.3	54 000	232 340	1 570	19.38
20	82	24.17	.570	8.890	.015	1 559.8	79.9	8.03	1.82	156.0	102 400	207 980	1 307	15.65
	73	21.37	.430	8.750	.015	1 466.5	75.9	8.28	1.88	146.7	64 900	195 540	1 307	16.13
	69	20.26	.520	8.145	.015	1 268.9	51.2	7.91	1.59	126.9	88 200	169 190	1 307	15.51
	64.5	18.86	.450	8.075	.015	1 222.1	49.8	8.05	1.62	122.2	69 400	162 950	1 307	15.77
	59.5	17.36	.375	8.000	.015	1 172.2	48.3	8.22	1.66	117.2	50 000	156 290	1 307	16.09
18	59.0	17.40	.495	7.675	.016	883.3	39.1	7.12	1.50	98.1	78 000	130 860	1 177	13.93
	54.5	15.87	.410	7.590	.016	842.0	37.7	7.28	1.54	93.6	57 500	124 740	1 177	14.24
	52.0	15.24	.375	7.555	.016	825.0	37.1	7.36	1.56	91.7	49 200	122 220	1 177	14.38
	49.0	14.25	.320	7.500	.016	798.3	36.2	7.48	1.59	88.7	36 700	118 260	1 177	14.62
15	71.[illegible]	20.95	.520	7.500	.020	796.2	61.3	6.16	1.71	106.2	77 900	141 540	980	11.85
	64.0	18.81	.605	7.195	.020	664.9	41.9	5.95	1.49	88.6	93 900	118 200	980	11.51
	54.5	15.88	.410	7.000	.020	610.0	38.3	6.20	1.55	81.3	54 800	108 450	980	12.00
	46.0	13.52	.440	6.810	.020	484.8	25.2	5.99	1.36	64.6	60 000	86 180	980	11.66
	41.0	12.02	.340	6.710	.020	456.7	24.0	6.16	1.41	60.9	39 900	81 180	980	12.00
	38.5	11.27	.290	6.660	.020	442.6	23.4	6.27	1.44	59.0	30 100	78 680	980	12.20
12	36.5	10.61	.310	6.300	.025	269.2	21.3	5.04	1.42	44.9	32 200	59 830	785	9.67
	32.0	9.44	.335	6.205	.025	228.5	16.0	4.92	1.30	38.1	35 800	50 770	785	9.49
	28.5	8.42	.250	6.120	.025	216.2	15.3	5.07	1.35	36.0	22 200	48 050	785	9.77
10	28.5	8.34	.390	5.990	.029	134.6	12.1	4.02	1.21	26.9	39 800	35 880	654	7.67
	23.5	6.94	.250	5.850	.029	122.9	11.2	4.21	1.27	24.6	21 000	32 770	654	8.03
9	24.0	7.04	.365	5.555	.033	92.1	8.8	3.62	1.12	20.5	33 900	27 290	590	6 88
	20.5	6.01	.250	5.440	.033	85.1	8.2	3.76	1.17	18.9	20 100	25 220	590	7 16
8	19.5	5.78	.325	5.325	.037	60.6	6.7	3.24	1.08	15.1	26 900	20 200	522	6.11
	17.5	5.18	.250	5.250	.037	57 4	6.4	3.33	1.11	14.3	18 900	19 130	522	6.28

TABLE 151.—*Continued.*

Properties of Bethlehem I Beams

1923 Sections.

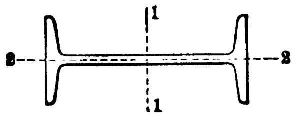
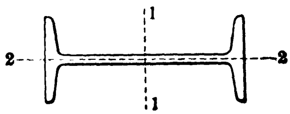
Depth of Beam.	Weight per Foot.	Area.	Thickness of Web.	Width of Flange.	 Moment of Inertia.		Radius of Gyration.		Section Modulus.	Maximum Safe Shear on Web.	Maximum Bending Moment @ 16,000 Lbs. per Sq. In.
					Axis 1–1.	Axis 2–2.	Axis 1–1.	Axis 2–2.	Axis 1–1.		
					I_1	I_2	r_1	r_2	S_1		M_1
In.	Lb.	In.²	In.	In.	In.⁴	In.⁴	In.	In.	In.³	Lb.	Ft.-Lb.
$30\frac{1}{8}$	129.0	37.52	.580	10.530	5566.5	177.5	12.18	2.18	369.6	119,100	493,000
30	121.0	35.36	.550	10.500	5213.6	164.3	12.14	2.16	347.6	107,300	463,500
$29\frac{7}{8}$	115.0	33.50	.530	10.480	4886.8	151.8	12.08	2.13	327.1	99,500	436,000
$28\frac{1}{8}$	113.0	32.98	.540	10.030	4285.5	142.3	11.40	2.08	304.8	103,300	406,400
28	106.0	30.93	.510	10.000	3993.8	130.9	11.36	2.06	285.3	92,300	380,400
$27\frac{7}{8}$	100.0	29.18	.490	9.980	3723.4	120.2	11.30	2.03	267.1	85,000	356,000
$26\frac{1}{8}$	98.0	28.47	.500	9.530	3200.9	110.6	10.60	1.97	245.1	88,500	327,000
26	91.0	26.55	.470	9.500	2962.8	100.9	10.56	1.95	227.9	78,300	304,000
$25\frac{7}{8}$	85.5	24.89	.450	9.480	2742.2	91.6	10.50	1.92	211.9	71,600	282,500
$24\frac{3}{32}$	104.5	30.63	.550	9.775	2967.7	132.9	9.84	2.08	246.4	104,300	328,500
24	99.5	29.15	.525	9.750	2811.7	124.8	9.82	2.07	234.3	95,900	312,400
$23\frac{29}{32}$	95.5	27.79	.505	9.730	2663.1	117.1	9.79	2.05	222.8	89,300	297,000
$22\frac{1}{8}$	71.5	20.88	.420	8.535	1705.2	65.8	9.04	1.78	154.2	62,500	205,600
$22\frac{1}{16}$	68.5	20.04	.405	8.520	1629.3	62.3	9.02	1.76	147.7	58,200	197,000
22	65.5	19.08	.385	8.500	1549.5	58.8	9.01	1.76	140.9	52,600	188,000
$18\frac{1}{8}$	74.0	21.61	.440	8.770	1238.0	82.9	7.57	1.96	136.6	66,100	182,500
18	69.0	20.20	.420	8.750	1142.5	75.6	7.52	1.93	126.9	60,800	169,000
$17\frac{7}{8}$	64.5	18.79	.400	8.730	1048.5	68.4	7.47	1.91	117.3	55,600	156,400

TABLE 152

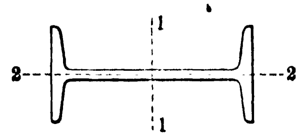
Properties of Bethlehem Girder Beams.



Depth of Beam.	Weight per Foot.	Area.	Thickness of Web.	Width of Flange.	Moment of Inertia. Axis 1-1. I_1	Moment of Inertia. Axis 2-2. I_2	Radius of Gyration. Axis 1-1. r_1	Radius of Gyration. Axis 2-2. r_2	Section Modulus. Axis 1-1. S_1	Maximum Safe Shear on Web	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. M_1
In.	Lb.	In.^{2}	In.	In.	In.^{4}	In.^{4}	In.	In.	In.^{3}	Lb.	Ft.-Lb.
30⅛	200.0	58.52	.76	15.04	9148.8	628.5	12.50	3.28	607.5	193,200	810,000
30	190.0	55.52	.72	15.00	8651.1	589.4	12.48	3.26	576.7	176,400	765,200
29⅞	181.0	52.82	.69	14.97	8181.0	552.0	12.45	3.23	547.6	163,700	730,000
28⅛	175.0	51.02	.70	14.29	6988.7	496.2	11.70	3.12	497.1	164,800	662,700
28	165.0	48.19	.66	14.25	6577.9	462.8	11.68	3.10	469.9	149,100	626,000
26⅛	160.0	46.85	.67	13.79	5576.6	432.8	10.91	3.04	427.0	149,500	569,400
26	151.0	44.16	.63	13.75	5237.1	402.7	10.89	3.02	402.9	134,900	537,000
25⅞	144.0	41.99	.61	13.73	4930.6	375.0	10.84	2.99	381.0	127,300	508,000
24⅛	149.0	43.57	.65	13.29	4451.1	383.3	10.11	2.97	369.1	138,200	492,000
24	141.0	41.02	.61	13.25	4174.2	356.4	10.09	2.95	347.9	124,600	462,000
23⅞	133.0	38.71	.58	13.22	3912.4	330.7	10.05	2.92	327.7	114,400	437,000
24⅛	129.0	37.74	.58	12.29	3844.8	278.2	10.09	2.72	318.8	114,800	425,000
24	121.0	35.30	.54	12.25	3585.3	256.9	10.08	2.70	298.8	101,400	398,400
23⅞	114.0	33.12	.51	12.22	3340.6	236.7	10.04	2.67	279.8	91,400	373,000
20⅛	149.0	43.44	.69	12.78	3106.6	384.5	8.46	2.97	308.8	138,100	417,700
20	142.0	41.31	.66	12.75	2932.3	360.9	8.43	2.96	293.2	129,200	391,000
19⅞	135.0	39.18	.63	12.72	2760.6	337.6	8.39	2.94	277.7	120,500	370,200
20⅛	120.0	34.95	.59	12.03	2505.5	260.1	8.47	2.73	249.1	109,800	332,000
20	113.0	32.90	.56	12.00	2340.2	240.8	8.43	2.71	234.0	101,000	312,000
19⅞	107.0	31.06	.54	11.98	2184.0	222.3	8.39	2.68	219.7	95,100	293,000
18⅛	100.0	29.25	.52	11.54	1725.7	202.6	7.68	2.63	190.5	86,300	254,000
18	93.0	27.14	.48	11.50	1593.4	185.1	7.66	2.61	177.0	76,000	236,000
17⅞	87.5	25.40	.46	11.48	1472.8	168.9	7.61	2.58	164.7	70,700	219,600

TABLE 152, *Continued.*

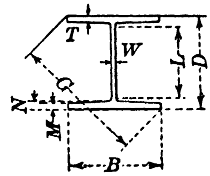
Properties of Bethlehem Girder Beams.

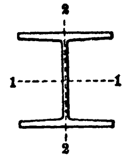


Depth of Beam.	Weight per Foot.	Area.	Thickness of Web.	Width of Flange.	Moment of Inertia. Axis 1-1. I_1	Moment of Inertia. Axis 2-2. I_2	Radius of Gyration. Axis 1-1. r_1	Radius of Gyration. Axis 2-2. r_2	Section Modulus. Axis 1-1. S_1	Maximum Safe Shear on Web.	Maximum Bending Moment @ 16,000 Lbs. per Sq. In. M_1
In.	Lb.	In.²	In.	In.	In.⁴	In.⁴	In.	In.	In.³	Lb.	Ft.-Lb.
15⅛	147.0	42.73	.83	11.78	1666.2	347.3	6.24	2.85	220.4	125,500	293,900
15	141.0	40.86	.80	11.75	1577.7	328.3	6.21	2.83	210.4	120,000	280,500
14⅞	135.0	39.01	.77	11.72	1490.7	309.5	6.18	2.82	200.4	114,600	267,000
15⅛	111.0	32.40	.64	11.29	1306.3	231.2	6.35	2.67	172.8	96,800	230,400
15	105.0	30.45	.60	11.25	1218.2	214.3	6.32	2.65	162.4	90,000	216,600
14⅞	99.0	28.65	.57	11.22	1134.7	198.4	6.29	2.63	152.5	84,800	203,400
15⅛	80.5	23.44	.48	10.79	968.5	143.0	6.43	2.47	128.1	69,700	170,900
15	74.0	21.55	.44	10.75	883.8	128.9	6.40	2.45	117.8	61,100	157,100
14⅞	69.0	19.96	.42	10.73	806.4	115.8	6.36	2.41	108.4	56,600	144,500
12⅛	76.5	22.29	.51	10.29	589.0	132.1	5.14	2.43	97.2	61,800	129,600
12	70.5	20.57	.47	10.25	538.4	119.7	5.12	2.41	89.7	56,400	119,600
11⅞	66.0	19.11	.45	10.23	491.7	108.3	5.07	2.38	82.8	53,500	110,400
12⅛	61.0	17.77	.41	10.03	479.9	95.8	5.20	2.32	79.2	49,100	105,600
12	55.5	16.21	.38	10.00	431.8	84.9	5.16	2.29	72.0	43,800	95,900
11 29/32	51.5	15.07	.36	9.98	396.9	76.9	5.13	2.26	66.6	40,300	88,800
10⅛	50.0	14.51	.36	9.04	275.5	66.4	4.36	2.14	54.4	36,400	72,600
10	44.5	13.03	.32	9.00	244.7	58.2	4.33	2.11	48.9	31,100	65,300
9 29/32	41.5	12.12	.31	8.99	223.8	52.6	4.30	2.08	45.2	29,500	60,200
9⅛	43.5	12.62	.35	8.54	193.8	51.3	3.92	2.02	42.5	31,900	56,700
9	38.5	11.23	.31	8.50	170.3	44.4	3.89	1.99	37.9	27,900	50,500
8 15/16	36.0	10.55	.29	8.48	158.9	41.0	3.88	1.97	35.5	25,300	47,400
8⅛	37.0	10.77	.33	8.03	131.1	38.7	3.49	1.90	32.3	26,800	43,000
8	33.0	9.57	.30	8.00	114.2	33.2	3.45	1.86	28.6	24,000	38,000
7 15/16	31.0	9.01	.29	7.99	106.2	30.5	3.43	1.81	26.7	23,000	35,600

TABLE 153

Properties of Bethlehem H Columns





Depth	Weight per Foot	Nominal Flange Thickness	Width of Flange	Thickness of Web					Area of Section	Moment of Inertia Axis 1–1	Moment of Inertia Axis 2–2	Radius of Gyration Axis 1–1	Radius of Gyration Axis 2–2	Section Modulus Axis 1–1	Section Modulus Axis 2–2
D		T	B	W	M	N	G	L		I_1	I_2	r_1	r_2	S_1	S_2
In.	Lb.	In.	In.	In.	In.	In.	In.	In.	In.²	In.⁴	In.⁴	In.	In.	In.³	In.³
							14″ H Columns								
13 3/4	84.0	11/16	13.92	.43	.620	.755	19 5/8		24.46	884.9	294.5	6.01	3.47	128.7	42.3
13 7/8	92.0	3/4	13.96	.47	.683	.817	19 3/4		26.76	976.8	325.4	6.04	3.49	140.8	46.6
14	100.0	13/16	14.00	.51	.745	.880	19 13/16		29.06	1 070.6	356.9	6.07	3.50	153.0	51.0
14 1/8	107.5	7/8	14.04	.55	.808	.942	19 15/16		31.38	1 166.6	387.8	6.10	3.52	165.2	55.2
14 1/4	115.5	15/16	14.08	.59	.870	1.005	20 1/16		33.70	1 264.5	420.3	6.13	3.53	177.5	59.7
14 3/8	123.5	1	14.12	.63	.933	1.067	20 3/16		36.04	1 364.6	453.4	6.16	3.55	189.9	64.2
14 1/2	131.5	1 1/16	14.16	.67	.995	1.130	20 1/4		38.38	1 466.7	486.9	6.18	3.56	202.3	68.8
14 5/8	139.0	1 1/8	14.19	.70	1.058	1.192	20 3/8		40.59	1 568.4	519.7	6.21	3.58	214.5	73.3
14 3/4	147.0	1 3/16	14.23	.74	1.120	1.255	20 1/2		42.95	1 674.7	554.4	6.24	3.59	227.1	77.9
14 7/8	155.0	1 1/4	14.27	.78	1.183	1.317	20 5/8		45.33	1 783.3	589.5	6.27	3.61	239.8	82.6
15	163.0	1 5/16	14.31	.82	1.245	1.380	20 3/4		47.71	1 894.0	626.1	6.30	3.62	252.5	87.5
15 1/8	171.5	1 3/8	14.35	.86	1.308	1.442	20 7/8		50.11	2 007.0	662.3	6.33	3.64	265.4	92.3
15 1/4	179.5	1 7/16	14.39	.90	1.370	1.505	21		52.51	2 122.3	699.0	6.36	3.65	278.3	97.2
15 3/8	187.5	1 1/2	14.43	.94	1.433	1.567	21 1/8	L is constant = 11.06″	54.92	2 239.8	736.3	6.39	3.66	291.4	102.1
15 1/2	196.0	1 9/16	14.47	.98	1.495	1.630	21 1/4		57.35	2 359.7	774.2	6.41	3.67	304.5	107.0
15 5/8	204.5	1 5/8	14.51	1.02	1.558	1.692	21 3/8		59.78	2 481.9	812 6	6.44	3.69	317.7	112.0
15 3/4	212.0	1 11/16	14.54	1.05	1.620	1.755	21 7/16		62.07	2 603.3	849.8	6.48	3.70	330.6	116.9
15 7/8	220.5	1 3/4	14.58	1.09	1.683	1.817	21 9/16		64.52	2 730.2	889.3	6.51	3.71	344.0	122.0
16	228.5	1 13/16	14.62	1.13	1.745	1.880	21 11/16		66.98	2 859.6	929.4	6.53	3.73	357.5	127.1
16 1/8	237.0	1 7/8	14.66	1.17	1.808	1.942	21 13/16		69.45	2 991.5	970.0	6.56	3 74	371.0	132.3
16 1/4	245.5	1 15/16	14.70	1.21	1.870	2.005	21 15/16		71.94	3 125.8	1 011.3	6.59	3.75	384.7	137.6
16 3/8	254.0	2	14.74	1.25	1.933	2.067	22 1/16		74.43	3 262.7	1 053.2	6.62	3.76	398.5	142.9
16 1/2	262.5	2 1/16	14.78	1.29	1.995	2.130	22 3/16		76.93	3 402.1	1 095.6	6.65	3.77	412.4	148.3
16 5/8	271.0	2 1/8	14.82	1.33	2.058	2.192	22 5/16		79.44	3 544.1	1 138.7	6.68	3.79	426.4	153.7
16 3/4	279.5	2 3/16	14.86	1.37	2.120	2.255	22 7/16		81.97	3 688.8	1 182.4	6.71	3.80	440.5	159.1
16 7/8	288.5	2 1/4	14.90	1.41	2.183	2.317	22 9/16		84.50	3 836.1	1 226.7	6.74	3.81	454.7	164.7
							12″ H Columns								
11 3/4	65.5	5/8	11.92	.39	.567	683	16 3/4		19.00	499.0	168.6	5.13	2.98	84.9	28.3
11 7/8	72.5	11/16	11.96	.43	.630	.745	16 7/8		20.96	556.6	188.2	5.15	3.00	93.7	31.5
12	79.0	3/4	12.00	.47	.692	.808	17		22.94	615.6	208.1	5.18	3.01	102.6	34.7
12 1/8	85.5	13/16	12.04	.51	.755	.870	17 1/8		24.92	676.1	228.5	5.21	3.03	111.5	37.9
12 1/4	92.5	7/8	12.08	.55	.817	.933	17 1/4	L is constant = 9.21″	26 92	738.1	249.2	5.24	3.04	120.5	41.3
12 3/8	99.5	15/16	12.12	.59	.880	.995	17 3/8		28.92	801.7	270.1	5.27	3.06	129.6	44.6
12 1/2	106.0	1	12.16	.63	.942	1.058	17 7/16		30.94	866.8	291.7	5.30	3.07	138.6	48.0
12 5/8	113.0	1 1/16	12.20	.67	1.005	1.120	17 9/16		32 96	933.4	313.6	5.33	3.08	147.9	51.4
12 3/4	119.5	1 1/8	12.23	.70	1.067	1.183	17 11/16		34.87	1 000.0	335.0	5 36	3.10	156.9	54.8
12 7/8	126.5	1 3/16	12.27	.74	1.130	1.245	17 13/16		36.91	1 069.8	357.7	5.38	3.11	166.2	58.3
13	133.5	1 1/4	12.31	.78	1.192	1.308	17 15/16		38.97	1 141.3	380.7	5.41	3.13	175.6	61.9

TABLE 153.—*Continued*

Properties of Bethlehem H Columns

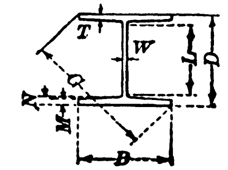
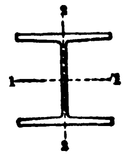
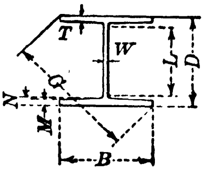
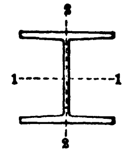
Depth	Weight per Foot	Nominal Flange Thickness	Width of Flange	Thickness of Web					Area	 Moment of Inertia		Radius of Gyration		Section Modulus	
										Axis 1-1	Axis 2-2	Axis 1-1	Axis 2-2	Axis 1-1	Axis 2-2
D		T	B	W	M	N	G	L		I_1	I_2	r_1	r_2	S_1	S_2
In.	Lb.	In.	In.	In.	In.	In.	In.	In.	In.^{2}	In.^{4}	In.^{4}	In.	In.	In.^{3}	In.^{3}
								12" H Columns							
13⅛	140.5	1 5/16	12.35	.82	1.255	1.370	18	L is constant = 9.21"	41.03	1 214.5	404.1	5.44	3.14	185.0	65.4
13¼	147.5	1⅜	12.39	.86	1.317	1.433	18⅛		43.10	1 289.4	428.0	5.47	3.15	194.6	69.1
13⅜	154.5	1 7/16	12.43	.90	1.380	1.495	18¼		45.19	1 366.0	452.2	5.50	3.16	204.3	72.8
13½	162.0	1½	12.47	.94	1.442	1.558	18⅜		47.28	1 444.3	477.0	5.53	3.18	214.0	76.5
								10" H Columns							
9⅞	49.5	9/16	9.97	.36	.514	.611	14 1/16	L is constant = 7.67"	14.37	263.5	89.1	4.28	2.49	53.4	17.9
10	55.0	⅝	10.00	.39	.577	.673	14 3/16		15.91	296.8	100 4	4.32	2.51	59.4	20.1
10⅛	60.5	11/16	10.04	.43	.639	.736	14 5/16		17.57	331.9	112.2	4.35	2.53	65.6	22.3
10¼	66.0	¾	10.08	.47	.702	.798	14⅜		19.23	368.0	124.2	4.37	2 54	71.8	24.6
10⅜	72.0	13/16	10.12	.51	.764	.861	14½		20.91	405.2	136.5	4.40	2.56	78.1	27.0
10½	77.5	⅞	10.16	.55	.827	.923	14⅝		22.59	443.6	149.1	4.43	2.57	84.5	29.4
10⅝	83.5	15/16	10.20	.59	.889	.986	14¾		24.29	483.0	162.0	4.46	2.58	90.9	31.8
10¾	89.0	1	10.24	.63	.952	1.048	14⅞		25.99	523.5	175.1	4.49	2.60	97.4	34.2
10⅞	95.0	1 1/16	10.28	.67	1.014	1.111	15		27.71	565.2	188.6	4.52	2.61	103.9	36.7
11	100.5	1⅛	10.31	.70	1.077	1.173	15⅛		29.32	607.0	201.7	4.55	2.62	110.4	39.1
11⅛	106.5	1 3/16	10.35	.74	1.139	1.236	15 3/16		31.06	651.0	215.6	4.58	2.64	117.0	41.7
11¼	112.0	1¼	10.39	.78	1.202	1.298	15 5/16		32.80	696.2	229.9	4.61	2.65	123.8	44.3
11⅜	118.0	1 5/16	10.43	.82	1.264	1.361	15 7/16		34.55	742.7	244.4	4.64	2.66	130.6	46.9
11½	124.0	1⅜	10.47	.86	1.327	1.423	15 9/16		36.32	790.4	259.3	4.67	2.67	137.5	49.5
								8" H Columns							
7⅞	32.0	7/16	8.00	.31	.399	.476	11¼	L is constant = 6.14"	9.17	105.7	35.8	3.40	1.98	26.9	8.9
8	35.0	½	8.00	.31	.462	.538	11⅜		10.17	121.5	41.1	3.46	2.01	30.4	10.3
8⅛	39.5	9/16	8.04	.35	.524	.601	11 7/16		11.50	139.5	47 2	3.48	2.03	34.3	11.7
8¼	44.0	⅝	8.08	.39	.587	.663	11 9/16		12.83	158.3	53.4	3.51	2.04	38.4	13.2
8⅜	48.5	11/16	8.12	.43	.649	.726	11 11/16		14.18	177.7	59.8	3.54	2.05	42.4	14.7
8½	53.0	¾	8.16	.47	.712	.788	11 13/16		15.53	197.8	66.3	3.57	2.07	46.5	16.3
8⅝	58.0	13/16	8.20	.51	.774	.851	12		16.90	218.6	73.1	3.60	2.08	50.7	17.8
8¾	62.0	⅞	8.24	.55	.837	.913	12 1/16		18.27	240.2	80.0	3.63	2.09	54.9	19.4
8⅞	67.5	15/16	8.28	.59	.899	.976	12⅛		19.66	262.5	87.1	3.65	2.11	59.2	21.0
9	72.0	1	8.32	.63	.962	1.038	12¼		21.05	285.6	94.4	3.68	2.12	63.5	22.7
9⅛	77.0	1 1/16	8.36	.67	1.024	1.101	12⅜		22.46	309.5	101.9	3.71	2.13	67.8	24.4
9¼	81.5	1⅛	8.39	.70	1.087	1.163	12½		23.78	333.5	109.2	3 75	2.14	72.1	26.0
9⅜	86.0	1 3/16	8.43	.74	1.149	1.226	12⅝		25.20	359.0	117.2	3.77	2.16	76.6	27.8
9½	91.0	1¼	8.47	.78	1.212	1.288	12¾		26.64	385.3	125.1	3.80	2.17	81.1	29.6

TABLE 153.—*Continued.*

PROPERTIES OF BETHLEHEM H COLUMNS.

Supplementary Sections.



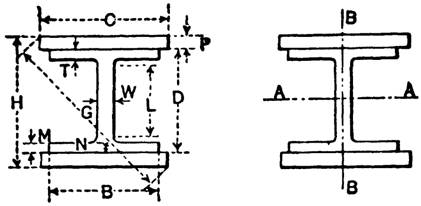


Depth.	Weight per Foot.	Nominal Flange Thickness.	Width of Flange.	Thickness of Web.					Area of Section.	Moment of Inertia.		Radius of Gyration.		Section Modulus.	
										Axis 1–1.	Axis 2–2.	Axis 1–1.	Axis 2–2.	Axis 1–1.	Axis 2–2.
D		T	B	W	M	N	G	L		I_1	I_2	r_1	r_2	S_1	S_2
In.	Lb.	In.	In.	In.	In.	In.	In.	In.	In.²	In.⁴	In.⁴	In.	In.	In.³	In.³
							14″ H Columns								
$13\frac{3}{8}$	43.0	$\frac{1}{2}$	8.00	.31	.491	.567	$15\frac{9}{16}$	$11\frac{1}{16}$	12.28	397.6	43.6	5.69	1.88	59.5	10.9
$13\frac{1}{2}$	48.0	$\frac{9}{16}$	8.04	.35	.553	.630	$15\frac{11}{16}$	$11\frac{1}{16}$	13.82	450.9	49.7	5.71	1.90	66.8	12.4
$13\frac{5}{8}$	53.5	$\frac{5}{8}$	8.08	.39	.616	.692	$15\frac{13}{16}$	$11\frac{1}{16}$	15.37	505.6	56.0	5.74	1.91	74.2	13.9
$13\frac{3}{4}$	58.5	$\frac{11}{16}$	8.12	.43	.678	.755	16	$11\frac{1}{16}$	16.93	561.6	62.4	5.76	1.92	81.7	15.4
$13\frac{1}{2}$	55.0	$\frac{9}{16}$	10.00	.35	.533	.630	$16\frac{13}{16}$	$11\frac{1}{16}$	15.95	540.4	93.1	5.82	2.42	80.1	18.6
$13\frac{5}{8}$	61.5	$\frac{5}{8}$	10.04	.39	.596	.692	$16\frac{15}{16}$	$11\frac{1}{16}$	17.75	606.3	104.8	5.84	2.43	89.0	20.9
$13\frac{3}{4}$	67.5	$\frac{11}{16}$	10.08	.43	.658	.755	$17\frac{1}{16}$	$11\frac{1}{16}$	19.55	673.7	116.8	5.87	2.44	98.0	23.2
$13\frac{7}{8}$	73.5	$\frac{3}{4}$	10.12	.47	.721	.817	$17\frac{3}{16}$	$11\frac{1}{16}$	21.37	742.7	129.0	5.90	2.46	107.1	25.5
$13\frac{5}{8}$	69.0	$\frac{5}{8}$	12.00	.39	.576	.692	$18\frac{1}{8}$	$11\frac{1}{16}$	20.04	704.0	174.7	5.93	2.95	103.3	29.1
$13\frac{3}{4}$	76.0	$\frac{11}{16}$	12.04	.43	.639	.755	$18\frac{1}{4}$	$11\frac{1}{16}$	22.09	782.9	194.7	5.95	2.97	113.9	32.3
$13\frac{7}{8}$	83.0	$\frac{3}{4}$	12.08	.47	.701	.817	$18\frac{3}{8}$	$11\frac{1}{16}$	24.15	863.6	215.0	5.98	2.98	124.5	35.6
14	90.0	$\frac{13}{16}$	12.12	.51	.764	.880	$18\frac{1}{2}$	$11\frac{1}{16}$	26.22	946.1	235.8	6.01	3.00	135.2	38.9
							12″ H Columns								
$11\frac{1}{2}$	40.5	$\frac{1}{2}$	8.00	.31	.481	.558	14	$9\frac{3}{16}$	11.55	280.1	42.8	4.92	1.92	48.7	10.7
$11\frac{5}{8}$	45.5	$\frac{9}{16}$	8.04	.35	.543	.620	$14\frac{1}{8}$	$9\frac{3}{16}$	13.02	318.8	48.8	4.95	1.94	54.8	12.1
$11\frac{3}{4}$	50.5	$\frac{5}{8}$	8.08	.39	.606	.683	$14\frac{1}{4}$	$9\frac{3}{16}$	14.49	358.5	55.1	4.97	1.95	61.0	13.6
$11\frac{7}{8}$	55.0	$\frac{7}{16}$	8.12	.43	.668	.745	$14\frac{3}{8}$	$9\frac{3}{16}$	15.98	399.3	61.5	5.00	1.96	67.3	15.2
$11\frac{5}{8}$	52.5	$\frac{9}{16}$	10.00	.35	.524	.620	$15\frac{5}{16}$	$9\frac{3}{16}$	15.11	383.2	91.5	5.04	2.46	65.9	18.3
$11\frac{3}{4}$	58.0	$\frac{5}{8}$	10.04	.39	.586	.683	$15\frac{7}{16}$	$9\frac{3}{16}$	16.83	431.3	103.2	5.06	2.48	73.4	20.5
$11\frac{7}{8}$	64.0	$\frac{11}{16}$	10.08	.43	.649	.745	$15\frac{9}{16}$	$9\frac{3}{16}$	18.56	480.6	115.1	5.09	2.49	80.9	22.8
12	70.0	$\frac{3}{4}$	10.12	.47	.711	.808	$15\frac{11}{16}$	$9\frac{3}{16}$	20.30	531.3	127.3	5.12	2.50	88.5	25.2
							10″ H Columns								
$9\frac{5}{8}$	33.5	$\frac{7}{16}$	8.00	.28	.408	.486	$12\frac{1}{2}$	$7\frac{11}{16}$	9.60	166.2	36.6	4.16	1.95	34.5	9.14
$9\frac{3}{4}$	38.0	$\frac{1}{2}$	8.03	.31	.471	.548	$12\frac{5}{8}$	$7\frac{11}{16}$	10.89	192.0	42.4	4.20	1.97	39.4	10.56
$9\frac{7}{8}$	42.5	$\frac{9}{16}$	8.07	.35	.533	.611	$12\frac{3}{4}$	$7\frac{11}{16}$	12.29	219.4	48.5	4.23	1.99	44.4	12.02
10	47.5	$\frac{5}{8}$	8.11	.39	.596	.673	$12\frac{7}{8}$	$7\frac{11}{16}$	13.70	247.6	54.8	4.25	2.00	49.5	13.52
							8″ H Columns								
$7\frac{3}{4}$	23.5	$\frac{3}{8}$	6.50	.25	.351	.413	$10\frac{1}{8}$	$6\frac{1}{8}$	6.72	74.6	16.8	3.33	1.58	19.2	5.17
$7\frac{7}{8}$	27.0	$\frac{7}{16}$	6.53	.28	.413	.476	$10\frac{1}{4}$	$6\frac{1}{8}$	7.76	88.2	20.0	3.37	1.60	22.4	6.11
8	30.5	$\frac{1}{2}$	6.56	.31	.476	.538	$10\frac{3}{8}$	$6\frac{1}{8}$	8.82	102.3	23.2	3.41	1.62	25.6	7.07
$8\frac{1}{8}$	34.5	$\frac{9}{16}$	6.60	.35	.538	.601	$10\frac{1}{2}$	$6\frac{1}{8}$	9.97	117.4	26.6	3.43	1.63	28.9	8.07
							6″ H Columns								
6	20.0	$\frac{3}{8}$	6.00	.25	.346	.404	$8\frac{1}{2}$	$4\frac{5}{8}$	5.81	38.7	13.0	2.58	1.50	12.9	4.34
$6\frac{1}{8}$	23.0	$\frac{7}{16}$	6.02	.27	.409	.466	$8\frac{9}{16}$	$4\frac{5}{8}$	6.69	45.9	15.4	2.62	1.52	15.0	5.12
$6\frac{1}{4}$	26.5	$\frac{1}{2}$	6.06	.31	.471	.529	$8\frac{11}{16}$	$4\frac{5}{8}$	7.69	53.9	18.1	2.65	1.53	17.3	5.96
$6\frac{3}{8}$	30.0	$\frac{9}{16}$	6.10	.35	.534	.591	$8\frac{13}{16}$	$4\frac{5}{8}$	8.70	62.4	20.8	2.68	1.55	19.6	6.82
$6\frac{1}{2}$	33.5	$\frac{5}{8}$	6.14	.39	.596	.654	$8\frac{15}{16}$	$4\frac{5}{8}$	9.72	71.2	23.6	2.71	1.56	21.9	7.69
$6\frac{5}{8}$	37.0	$\frac{11}{16}$	6.18	.43	.659	.716	$9\frac{1}{16}$	$4\frac{5}{8}$	10.76	80.4	26.6	2.73	1.57	24.3	8.59
$6\frac{3}{4}$	40.5	$\frac{3}{4}$	6.22	.47	.721	.779	$9\frac{3}{16}$	$4\frac{5}{8}$	11.80	90.1	29.6	2.76	1.58	26.7	9.52

TABLE 154.

PROPERTIES OF BETHLEHEM COMPOUND COLUMNS.

14⅛″ x 149 Lb. Special H Section.



Reënforced with Cover Plates

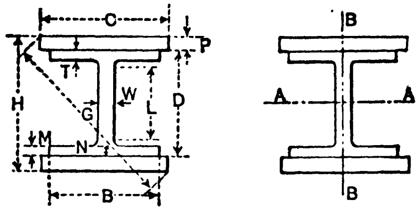
Depth.	Total Section.		Dimensions.				Moment of Inertia.		Radius of Gyration.		Section Modulus.	
	Weight.	Area	H Section.	Cover Plates. Width.	Cover Plates. Thickness.	G	Axis A-A	Axis B-B.	Axis A-A.	Axis B-B.	Axis A-A.	Axis B-B.
H				C	P		I_A	I_B	r_A	r_B	S_A	S_B
In.	Lb.	In.²	In.	In.	In.	In	In.⁴	In.⁴	In.	In.	In.³	In.³
16⅝	285.0	83.52		16	1¼	23 1/16	3737.7	1321.9	6.69	3.98	449.6	165.2
16¾	291.8	85.52	D	16	1 5/16	23 3/16	3876.9	1364.6	6.73	3.99	462.9	170.6
16⅞	298.6	87.52	14⅛	16	1⅜	23¼	4018.2	1407.3	6.78	4.01	476.2	175.9
17	305.4	89.52		16	1 7/16	23⅜	4161.7	1449.9	6.82	4.02	489.6	181.2
17⅛	312.2	91.52		16	1½	23 7/16	4307.2	1492.6	6.86	4.04	503.0	186.6
17¼	319.0	93.52		16	1 9/16	23½	4454.9	1535.3	6.90	4.05	516.5	191.9
17⅜	325.8	95.52	T	16	1⅝	23⅝	4604.8	1577.9	6.94	4.06	530.0	197.2
17½	332.6	97.52	⅞	16	1 11/16	23 11/16	4756.8	1620.6	6.98	4.08	543.6	202.6
17⅝	339.4	99.52		16	1¾	23 13/16	4911.0	1663.3	7.02	4.09	557.3	207.9
17¾	346.2	101.52		16	1 13/16	23⅞	5067.5	1705.9	7.07	4.10	571.0	213.2
17⅝	351.3	103.02	B	17	1¾	24½	5132.5	1901.6	7.06	4.30	582.4	223.7
17¾	358.5	105.15	14.90	17	1 13/16	24 9/16	5298.7	1952.8	7.10	4.31	597.0	229.7
17⅞	365.7	107.27		17	1⅞	24 11/16	5467.2	2003.9	7.14	4.32	611.7	235.8
18	373.0	109.40		17	1 15/16	24¾	5638.1	2055.1	7.18	4.33	626.5	241.8
18⅛	380.2	111.52	W	17	2	24⅞	5811.5	2106.3	7.22	4.35	641.3	247.8
18¼	387.4	113.65	1.41	17	2 1/16	24 15/16	5987.2	2157.5	7.26	4.36	656.1	253.8
18⅜	394.6	115.77		17	2⅛	25 1/16	6165.4	2208.7	7.30	4.37	671.1	259.8
18½	401.9	117.90	M	17	2 3/16	25⅛	6345.9	2259.8	7.34	4.38	686.0	265.9
18⅝	409.1	120.02	0.808	17	2¼	25 3/16	6529.0	2311.0	7.38	4.39	701.1	271.9
18¾	416.3	122.15		17	2 5/16	25 5/16	6714.5	2362.2	7.41	4.40	716.2	277.9
18⅝	424.4	124.52		18	2¼	25⅞	6832.6	2655.6	7.41	4.62	733.7	295.1
18¾	432.0	126.77		18	2 5/16	26	7029.0	2716.4	7.45	4.63	749.8	301.8
18⅞	439.7	129.02	N	18	2⅜	26 1/16	7228.1	2777.1	7.48	4.64	765.9	308.6
19	447.3	131.27	0.942	18	2 7/16	26 3/16	7429.8	2837 9	7.52	4.65	782.1	315.3
19⅛	455.0	133.52		18	2½	26¼	7634.2	2898.6	7.56	4.66	798.3	322.1
19¼	462.6	135.77		18	2 9/16	26⅜	7841.3	2959.4	7.60	4.67	814.7	328.8
19⅜	470.3	138.02	L	18	2⅝	26 7/16	8051.1	3020.1	7.64	4.68	831.1	335.6
19½	477.9	140.27	11.06	18	2 11/16	26 9/16	8263.6	3080.9	7.68	4.69	847.6	342.3
19⅝	485.6	142.52		18	2¾	26⅝	8478.9	3141.6	7.71	4.70	864.1	349.1

Columns composed of a 14″ × 148 lb. Special Column Section, reënforced with cover plates of width and thickness given in table. The total thickness, P, may be made of two or more plates, each of punchable thickness.

TABLE 154.—*Continued.*

Properties of Bethlehem Compound Columns.

14″ Special H Section.



Reënforced with Cover Plates.

Weight of H.	Depth. H.	Total Section. Wt.	Total Section. Area.	Dimensions. H Section.	Cover Plates. W'th. C	Cover Plates. Thickness. P	G.	Moment of Inertia. Axis A-A. I_A	Moment of Inertia. Axis B-B. I_B	Radius of Gyration. Axis A-A. r_A	Radius of Gyration. Axis B-B. r_B	Section Modulus. Axis A-A. S_A	Section Modulus. Axis B-B. S_B
	In.	Lb.	In.²	In.	In.	In.	In.	In.⁴	In.⁴	In.	In.	In.³	In.³
H 14 254.0 Lb.	17 1/8	294.8	86.43	D = 16 3/8	16	3/8	23 7/16	4104.5	1309.2	6.89	3.89	479.4	163.6
	17 1/4	301.6	88.43	T = 2	16	7/16	23 1/2	4252.2	1351.8	6.93	3.91	493.0	169.0
	17 3/8	308.4	90.43		16	1/2	23 5/8	4402.1	1394.5	6.98	3.93	506.7	174.3
	17 1/2	315.2	92.43	B = 14.74	16	9/16	23 11/16	4554.1	1437.2	7.02	3.94	520.5	179.6
	17 5/8	322.0	94.43	W = 1.25	16	5/8	23 13/16	4708.3	1479.8	7.06	3.96	534.3	185.0
	17 3/4	328.8	96.43		16	11/16	23 7/8	4864.8	1522.5	7.10	3.97	548.1	190.3
	17 7/8	335.6	98.43	M = 1.933	16	3/4	24	5023.4	1565.2	7.14	3.99	562.1	195.6
	18	342.4	100.43	N = 2.067	16	13/16	24 1/16	5184.3	1607.8	7.18	4.00	576.0	201.0
	18 1/8	349.2	102.43		16	7/8	24 3/16	5347.4	1650.5	7.23	4.01	590.1	206.3
	18 1/4	356.0	104.43	L = 11 1/16	16	15/16	24 1/4	5512.8	1693.2	7.27	4.03	604.1	211.6
H 14 288.5 Lb.	18 1/8	360.8	105.75		17	5/8	24 7/8	5463.7	1738.5	7.19	4.06	602.9	204.5
	18 1/4	368.0	107.87	D = 16 7/8	17	11/16	24 15/16	5639.4	1789.6	7.23	4.07	618.0	210.5
	18 3/8	375.2	110.00		17	3/4	25 1/16	5817.6	1840.8	7.27	4.09	633.2	216.6
	18 1/2	382.4	112.12		17	13/16	25 1/8	5998.2	~~1892.0~~	7.31	4.11	648.5	222.6
	18 5/8	389.7	114.25	T = 2 1/4	17	7/8	25 1/4	6181.2	1943.2	7.36	4.12	663.8	228.6
	18 3/4	396.9	116.37		17	15/16	25 5/16	6366.8	1994.3	7.40	4.14	679.1	234.6
	18 7/8	404.1	118.50	B = 14.90	17	1	25 3/8	6554.8	2045.5	7.44	4.15	694.5	240.6
	19	411.3	120.62		17	1 1/16	25 1/2	6745.3	2096.7	7.48	4.17	710.0	246.7
	19 1/8	418.6	122.75	W = 1.41	17	1 1/8	25 9/16	6938.3	2147.9	7.52	4.18	725.6	252.7
H 14 288.5 Lb.	19 1/8	426.2	125.00	--	18	1 1/8	26 1/4	7120.8	2320.2	7.55	4.31	744.7	257.8
	19 1/4	433.9	127.25		18	1 3/16	26 3/8	7327.9	2380.9	7.59	4.33	761.3	264.5
	19 3/8	441.5	129.50		18	1 1/4	26 7/16	7537.7	2441.7	7.63	4.34	778.1	271.3
	19 1/2	449.2	131.75	M = 2.183	18	1 5/16	26 9/16	7750.2	2502.4	7.67	4.36	794.9	278.0
	19 5/8	456.8	134.00		18	1 3/8	26 5/8	7965.5	2563.2	7.71	4.37	811.8	284.8
	19 3/4	464.5	136.25		18	1 7/16	26 3/4	8183.5	2623.9	7.75	4.39	828.7	291.5
	19 7/8	472.1	138.50	N = 2.317	18	1 1/2	26 13/16	8404.3	2684.7	7.79	4.40	845.7	298.3
	20	479.8	140.75		18	1 9/16	26 15/16	8627.9	2745.4	7.83	4.42	862.8	305.0
	20 1/8	487.4	143.00		18	1 5/8	27	8854.3	2806.2	7.87	4.43	879.9	311.8
	20 1/4	495.1	145.25	L = 11 1/16	18	1 11/16	27 1/8	9083.6	2866.9	7.91	4.44	897.1	318.5
	20 3/8	502.7	147.50		18	1 3/4	27 3/16	9315.7	2927.7	7.95	4.46	914.4	325.3

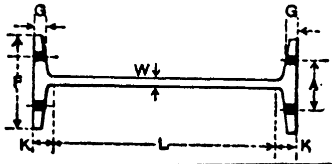
Supplementary H Columns

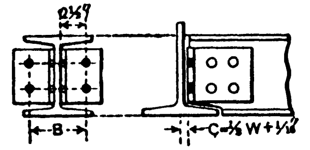
	19 3/4	493.2	144.77		18	2 13/16	26 3/4	8696.9	3202.4	7.75	4.70	880.7	355.8
	19 7/8	500.9	147.02		18	2 7/8	26 13/16	8917.7	3263.1	7.79	4.71	897.4	362.6

TABLE 155.

Elements of Bethlehem I-Beams and Girder Beams.

Elements of Bethlehem I Beams.

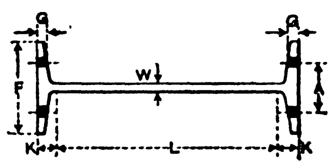


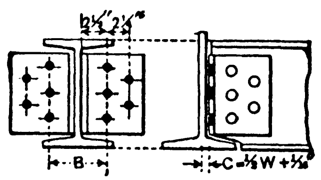


Depth of Beam, Inches.	Weight per Foot, Lbs.	F	W	L	K	G	A	B	C	Max. Rivet in Flange.
30	120.0	10 1/2	35/64	26 7/16	1 25/32	29/32	6 1/2	5 9/16	5/16	1
28	105.0	10	1/2	24 11/16	1 21/32	27/32	6	5 1/2	5/16	1
26	90.0	9 1/2	15/32	23	1 1/2	25/32	5 1/2	5 7/16	5/16	1
24	84.0	9 1/4	15/32	21	1 1/2	3/4	5 1/4	5 7/16	5/16	7/8
24	83.0	9 1/8	33/64	21 5/16	1 13/32	11/16	5 1/4	5 1/2	5/16	7/8
24	73.0	9	25/64	21 5/16	1 13/32	11/16	5 1/4	5 3/8	1/4	7/8
20	82.0	8 57/64	37/64	17 1/8	1 7/16	3/4	5	5 9/16	3/8	7/8
20	72.0	8 3/4	7/16	17 1/8	1 7/16	3/4	5	5 7/16	1/4	7/8
20	69.0	8 9/64	33/64	17 1/2	1 1/4	5/8	4 1/2	5 1/2	5/16	7/8
20	64.0	8 5/64	29/64	17 1/2	1 1/4	5/8	4 1/2	5 7/16	5/16	7/8
20	59.0	8	3/8	17 1/2	1 1/4	5/8	4 1/2	5 3/8	1/4	7/8
18	59.0	7 43/64	1/2	15 3/4	1 1/8	9/16	4 1/4	5 1/2	5/16	7/8
18	54.0	7 19/32	13/32	15 3/4	1 1/8	9/16	4 1/4	5 7/16	1/4	7/8
18	52.0	7 9/16	3/8	15 3/4	1 1/8	9/16	4 1/4	5 3/8	1/4	7/8
18	48.5	7 1/2	21/64	15 3/4	1 1/8	9/16	4 1/4	5 5/16	1/4	7/8

Depth of Beam, Inches.	Weight per Foot, Lbs.	F	W	L	K	G	A	B	C	Max. Rivet in Flange.
15	71.0	7 1/2	33/64	11 3/4	1 5/8	15/16	4 1/4	5 1/2	5/16	7/8
15	64.0	7 3/16	39/64	12 5/16	1 11/32	3/4	4	5 5/8	3/8	7/8
15	54.0	7	13/32	12 5/16	1 11/32	23/32	4	5 7/16	1/4	7/8
15	46.0	6 13/16	7/16	12 7/8	1 1/16	17/32	3 3/4	5 7/16	5/16	7/8
15	41.0	6 23/32	11/32	12 7/8	1 1/16	17/32	3 3/4	5 5/16	1/4	7/8
15	38.0	6 21/32	19/64	12 7/8	1 1/16	17/32	3 3/4	5 5/16	3/16	7/8
12	36.0	6 19/64	5/16	9 7/8	1 1/16	9/16	3 1/2	5 5/16	3/16	3/4
12	32.0	6 3/16	21/64	10 3/16	29/32	7/16	3 1/2	5 5/16	1/4	3/4
12	28.5	6 1/8	1/4	10 3/16	29/32	7/16	3 1/2	5 1/4	3/16	3/4
10	28.5	5 63/64	25/64	8 3/8	13/16	3/8	3 1/4	5 3/8	1/4	3/4
10	23.5	5 27/32	1/4	8 3/8	13/16	3/8	3 1/4	5 1/4	3/16	3/4
9	24.0	5 9/16	23/64	7 1/2	3/4	3/8	3	5 3/8	1/4	3/4
9	20.0	5 7/16	1/4	7 1/2	3/4	3/8	3	5 1/4	3/16	3/4
8	19.5	5 21/64	21/64	6 5/8	11/16	5/16	2 3/4	5 5/16	1/4	3/4
8	17.5	5 1/4	1/4	6 5/8	11/16	5/16	2 3/4	5 1/4	3/16	3/4

Elements of Bethlehem Girder Beams.





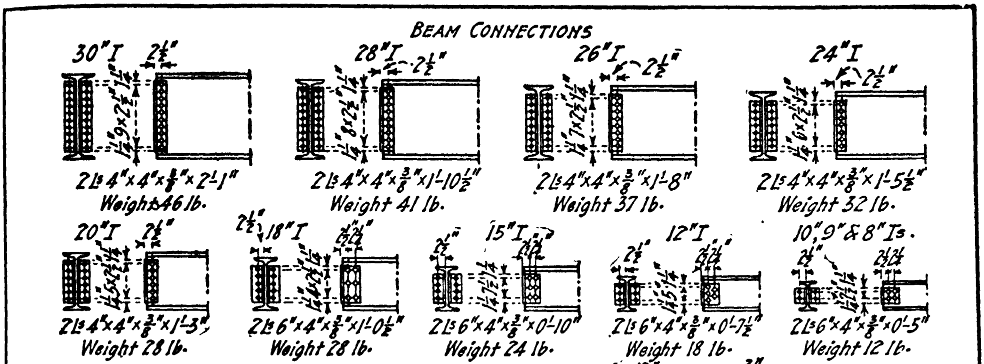
Depth of Beam, Inches.	Weight per Foot, Lbs.	F	W	L	K	G	A	B	C	Max Rivet in Flange.
30	200.0	15	3/4	25 3/16	2 13/32	1 1/8	11	5 3/4	7/16	1
30	180.0	13	11/16	25 3/16	2 13/32	1 7/32	9	5 11/16	7/16	1
28	180.0	14 11/32	11/16	23 3/8	2 5/16	1 3/32	10 1/4	5 11/16	7/16	1
28	165.0	12 1/2	21/32	23 3/8	2 5/16	1 3/16	8 1/2	5 11/16	3/8	1
26	160.0	13 19/32	5/8	21 5/8	2 3/16	1 1/16	9 1/2	5 5/8	3/8	1
26	150.0	12	5/8	21 5/8	2 3/16	1 1/8	8	5 5/8	3/8	1
24	140.0	13	19/32	20	2	31/32	9	5 5/8	3/8	1
24	120.0	12	17/32	20 1/4	1 7/8	29/32	8	5 1/2	5/16	1
20	140.0	12 1/2	41/64	15 11/16	2 3/32	1 1/8	8 1/2	5 5/8	3/8	1
20	112.0	12	35/64	16 3/8	1 13/16	7/8	8	5 9/16	5/16	1

Depth of Beam, Inches	Weight per Foot, Lbs.	F	W	L	K	G	A	B	C	Max Rivet in Flange.
18	92.0	11 1/2	37/64	14 3/4	1 5/8	25/32	7 1/2	5 1/2	5/16	1
15	140.0	11 3/4	51/64	10 1/8	2 7/16	1 9/32	7 3/4	5 13/16	7/16	1
15	104.0	11 1/4	19/32	11 1/8	1 15/16	15/16	7 1/4	5 5/8	3/8	1
15	73.0	10 1/2	7/16	12 1/16	1 15/32	11/16	6 1/2	5 7/16	1/4	1
12	70.0	10	15/32	9	1 1/2	3/4	6	5 7/16	5/16	1
12	55.0	9 3/4	3/8	9 1/2	1 1/4	19/32	6	5 3/8	1/4	1
10	44.0	9	5/16	7 3/4	1 1/8	17/32	5 1/2	5 5/16	3/16	7/8
9	38.0	8 1/2	19/64	6 7/8	1 1/16	15/32	5 1/4	5 5/16	3/16	7/8
8	32.5	8	19/64	6	1	7/16	5	5 5/16	3/16	7/8

Note. Elements for 1911 Standards.

TABLE 156.

Standard Connection Angles for Bethlehem I-Beams.



Spacing same in both legs of angles unless otherwise shown. All holes 13/16" Diam. for ¾" Diam. Rivets or Bolts.

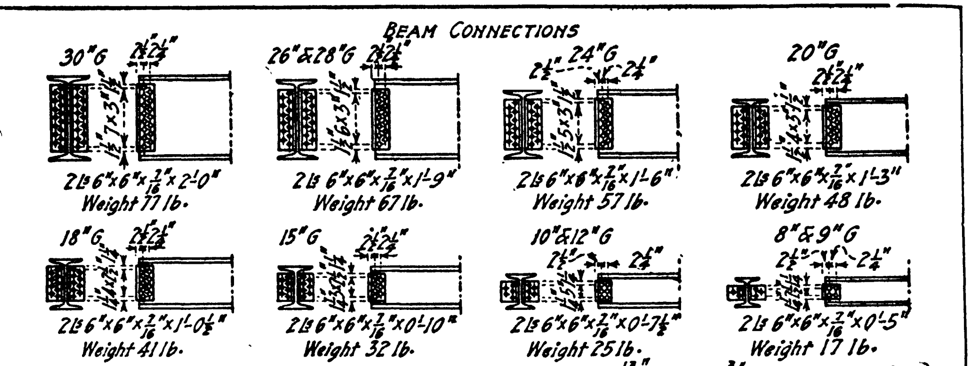
Minimum Spans on which the Above Connection Angles may be Used for Greatest Safe Uniformly Distributed Loads.

Depth of Beam, Inches.	Weight per Foot, Lbs.	Least Span, in Feet, for Various Conditions.								
		Rivets: Shearing 10,000 Lbs., Bearing 20,000 Lbs. per Square In.								Field Connection.
		Connection to Web of Beam.	Field Connection.	When Two Beams Frame Opposite Each Other to a Beam or Girder with a Web Thickness as Follows:						Rivet Shear, 8,000 Lbs. per Square Inch.
				5/16"	3/8"	7/16"	1/2"	9/16"	5/8"	
30	121.0	23.0	21.1	22.1	24.8	28.4	33.1	39.7	49.7	26.3
28	106.0	22.7	19.2	20.1	22.7	25.9	30.2	36.2	45.3	24.0
26	90.0	22.1	17.3	18.1	20.4	23.3	27.1	32.6	40.7	21.6
24	84.5	21.9	17.1	17.9	20.2	23.1	26.9	32.2	40.3	21.4
24	73.0	22.7	15.0	15.7	17.7	20.2	23.6	28.3	35.4	18.8
20	72.0	20.2	14.7	15.4	17.4	19.9	23.2	27.8	34.8	18.4
20	59.5	18.5	11.8	12.3	13.9	15.9	18.5	22.2	27.8	14.7
18	49.0	16.4	10.7	11.2	12.6	14.4	16.8	20.2	25.2	13.4
15	71.5	12.1	16.0	16.8	18.9	21.6	25.1	30.2	37.7	20.0
15	54.5	11.8	12.3	12.8	14.5	16.5	19.3	23.1	28.9	15.3
15	38.5	12.1	8.9	9.3	10.5	12.0	14.0	16.8	21.0	11.1
12	36.5	10.3	9.0	9.5	10.6	12.2	14.2	17.0	21.3	11.3
12	28.5	10.3	7.2	7.6	8.5	9.8	11.4	13.7	17.1	9.1
10	23.5	8.7	7.4	7.8	8.7	10.0	11.6	14.0	17.5	9.3
9	20.5	6.7	5.7	6.0	6.7	7.7	9.0	10.8	13.5	7.1
8	17.5	5.1	4.3	4.5	5.1	5.8	6.8	8.2	10.2	5.4

The greatest value given of the least span for any of the governing conditions is the minimum span for which the connection may be used.

TABLE 157.

Standard Connection Angles for Bethlehem Girder Beams.



Spacing same in both legs of angles unless otherwise shown. All holes 13/16" Diam. for 3/4" Diam. Rivets or Bolts.

Minimum Spans on which the Above Connection Angles May be Used for Greatest Safe Uniformly Distributed Loads.

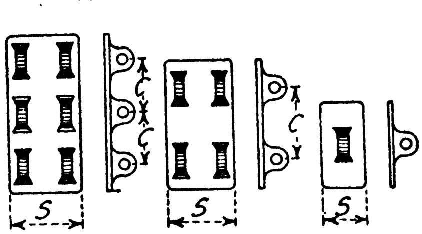
Depth of Beam, Inches.	Weight per Foot, Lbs.	Least Span, in Feet, for Various Conditions.								
		Rivet: Shearing 10,000 Lbs., Bearing 20,000 Lbs. per Sq. In.								Field Connection.
		Connection to Web of Beam.	Field Connection.	When Two Beams Frame Opposite Each Other to a Beam or Girder with a Web Thickness as Follows:						Rivet Shear, 8,000 Lbs. per Square Inch.
				9/16"	1/2"	7/16"	3/8"	5/16"	1/4"	
30	200.0	24.5	24.5	25.7	28.9	33.1	38.6	46.3	57.8	30.7
30	181.0	22.0	22.0	23.0	25.9	29.6	34.5	41.4	51.8	27.5
28	180.0	24.1	24.1	25.2	28.4	32.4	37.8	45.4	56.8	30.1
28	165.0	21.8	21.8	22.8	25.6	29.3	34.2	41.0	51.3	27.2
26	160.0	20.1	20.1	21.0	23.7	27.0	31.5	37.8	47.3	25.1
26	151.0	18.4	18.4	19.3	21.7	24.8	28.9	34.7	43.4	23.0
24	141.0	19.2	19.2	20.1	22.6	25.9	30.2	36.2	45.3	24.0
24	121.0	18.3	16.5	17.3	19.4	22.2	25.9	31.1	38.9	20.6
20	142.0	19.7	19.7	20.6	23.2	26.5	30.9	37.1	46.4	24.6
20	113.0	16.8	15.7	16.4	18.5	21.1	24.7	29.6	37.0	19.6
18	93.0	14.6	11.9	12.4	14.0	16.0	18.6	22.3	27.9	14.8
15	141.0	18.3	18.3	19.2	21.6	24.7	28.8	34.5	43.1	22.9
15	105.0	14.0	14.0	14.7	16.5	18.9	22.0	26.4	33.1	17.5
15	74.0	13.9	10.2	10.6	12.0	13.7	16.0	19.1	23.9	12.7
12	70.5	11.6	10.8	11.4	12.8	14.6	17.0	20.4	25.5	13.5
12	55.5	11.5	8.7	9.1	10.2	11.7	13.7	16.4	20.5	10.9
10	44.5	9.3	5.9	0.2	6.9	7.9	9.3	11.1	13.9	7.4
9	38.5	11.3	7.6	8.0	9.0	10.3	12.0	14.4	18.0	9.5
8	33.0	8.8	5.8	6.0	6.8	7.7	9.0	10.8	13.6	7.2

The greatest value given of the least span for any of the governing conditions is the minimum span for which the connection may be used.

TABLE 158.

Cast Iron Separators for Bethlehem Girder Beams and I-Beams.

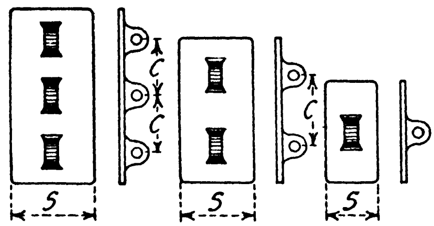
Bethlehem Girder Beams.



Separators for 18" to 30" beams are $\frac{5}{8}$" metal.
Separators for 8" to 15" beams are $\frac{1}{2}$" metal.

Beam.		Distances.		Bolts.		Weights.			
						Separators.		Bolts.	
Depth.	Weight per Foot.	C. to C. of Beams.	Width S.	C. to C.	Length.	For Width S.	For Each 1" Increase in S.	For Width S.	For Each 1" Increase in S.
In.	Lb.	In.	In.	In.	In.	Lb.	Lb.	Lb.	Lb.
Separators with Three Bolts.									
30	200.0	15¾	15	10	17½	73.0	4.50	7.7	.375
30	180.0	13¾	13	10	15½	64.5	4.50	7.0	.375
28	180.0	15	14¼	7½	16¾	65.0	4.15	7.4	.375
28	165.0	13¼	12⅝	7½	15	59.1	4.15	6.8	.375
26	160.0	14¼	13⅝	7½	16	59.0	3.85	7.1	.375
26	150.0	12¾	12⅛	7½	14½	53.0	3.85	6.6	.375
Separators with Two Bolts.									
24	140.0	13¾	13⅛	12½	15¼	50.0	3.50	4.6	.25
24	120.0	12¾	12¼	12½	14¼	47.0	3.50	4.3	.25
20	140.0	13	12⅜	10	14¾	39.0	2.80	4.5	.25
20	112.0	12½	12	10	14	38.0	2.80	4.3	.25
18	92.0	12	11½	10	13½	34.0	2.60	4.2	.25
15	140.0	12¼	11⅝	7½	14	22.0	1.50	4.3	.25
15	104.0	11¾	11⅛	7½	13½	22.0	1.60	4.2	.25
15	73.0	11	10½	7½	12½	21.0	1.60	4.0	.25
12	70.0	10½	10	5	12	17.5	1.30	3.8	.25
12	55.0	10⅜	10	5	11¾	17.5	1.30	3.8	.25
Separators with One Bolt.									
10	44.0	9½	9⅛	......	10¾	11.0	1.10	1.8	.125
9	38.0	9	8¾	......	10¼	10.0	1.00	1.7	.125
8	32.5	8½	8¼	......	9¾	8.0	.85	1.7	.125

Bethlehem I Beams.



Separators for 18" to 30" beams are $\frac{5}{8}$" metal.
Separators for 8" to 15" beams are $\frac{1}{2}$" metal.

Beam.		Distances.		Bolts.		Weights.			
						Separators.		Bolts.	
Depth.	Weight per Foot.	C. to C. of Beams.	Width S.	C. to C.	Length.	For Width S.	For Each 1" Increase in S.	For Width S.	For Each 1" Increase in S.
In.	Lb.	In.	In.	In.	In.	Lb.	Lb.	Lb.	Lb.
Separators with Three Bolts.									
30	120.0	11¼	10¾	10	12¾	50.1	4.50	6.0	.375
28	105.0	10⅝	10⅛	7½	12	43.9	4.15	5.7	.375
26	90.0	10⅛	9⅝	7½	11½	39.3	3.85	5.5	.375
Separators with Two Bolts.									
24	84.0	9¾	9¼	12½	11¼	35.1	3.65	3.6	.25
24	73.0	9⅝	9¼	12½	11	35.1	3.65	3.6	.25
20	72.0	9⅜	9	10	10¾	28.2	3.00	3.5	.25
20	59.0	8⅝	8¼	10	10	26.1	3.00	3.4	.25
18	48.5	8	7⅝	10	9¼	22.1	2.70	3.2	.25
15	71.0	8	7½	7½	9½	13.1	1.65	3.2	.25
15	54.0	7½	7	7½	9	12.3	1.65	3.1	.25
15	38.0	7¼	7	7½	8½	13.3	1.80	3.0	.25
12	36.0	6¾	6⅜	5	8	9.1	1.30	2.8	.25
12	28.5	6½	6¼	5	7¾	9.0	1.30	2.8	.25
Separators with One Bolt.									
10	23.5	6¼	6	......	7½	7.5	1.10	1.4	.125
9	20.0	5¾	5½	......	7	6.4	1.00	1.3	.125
8	17.5	5⅝	5⅜	......	6¾	5.5	.85	1.3	.125

Separators for 18 to 30 inch beams are ⅝ inch metal.
Separators for 8 to 15 inch beams are ½ inch metal.
All bolts ¾ inch diameter.

Note. 1911 Standard Separators.

TABLE 159.

Safe Loads, in Tons, and Deflections, in Inches, Bethlehem I-Beams.

Depth. In.	Weight. Lb.	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40	42
30	121	...	...	...	...	...	103	93	85	78	72	67	62	58	55	52	49	47	44
	*	...	...	...	...	...	.44	.39	.36	.33	.30	.28	.26	.25	.23	.22	.21	.20	.19
	Def.	...	...	...	...	...	*.18*	*.22*	*.27*	*.32*	*.37*	*.43*	*.50*	*.57*	*.64*	*.71*	*.80*	*.88*	*.97*
28	106	...	...	...	...	...	85	76	70	64	59	55	51	48	45	42	40	38	36
	*	...	...	...	...	...	.41	.37	.33	.31	.28	.26	.24	.23	.22	.20	.19	.19	.18
	Def.	...	...	...	...	...	*.19*	*.24*	*.29*	*.34*	*.40*	*.46*	*.53*	*.61*	*.68*	*.77*	*.85*	*.95*	*1.04*
26	91	...	...	...	...	...	68	61	56	51	47	44	41	38	36	34	32	31	29
	*	...	...	...	...	...	.38	.34	.31	.28	.26	.24	.23	.21	.20	.19	.18	.17	.16
	Def.	...	...	...	...	...	*.21*	*.25*	*.31*	*.37*	*.43*	*.50*	*.57*	*.65*	*.74*	*.83*	*.92*	*1.02*	*1.12*
24	84.5	...	...	88	76	66	59	53	48	44	41	38	35	33	31	29	28	26	...
	73.5	...	...	77	66	58	52	46	42	39	36	33	31	29	27	26	24	23	...
	*	...	...	.52	.45	.39	.35	.31	.29	.26	.24	.22	.21	.20	.19	.17	.17	.16	...
	Def.	...	...	*.10*	*.14*	*.18*	*.22*	*.28*	*.33*	*.40*	*.47*	*.54*	*.62*	*.71*	*.80*	*.89*	*1.00*	*1.10*	...
20	82	...	...	69	59	52	46	42	38	35	32	30	28	26	24	23	22	21	...
	72	...	...	65	56	49	43	39	36	33	30	28	26	24	23	22	21	20	...
	69	...	...	56	48	42	38	34	31	28	26	24	23	21	20	19	18	17	...
	64.5	...	...	54	47	41	36	33	30	27	25	23	22	20	19	18	17	16	...
	59.5	...	...	52	45	39	35	31	28	26	24	22	21	20	18	17	16	16	...
	*	...	...	.44	.37	.33	.29	.26	.24	.22	.20	.19	.17	.16	.15	.15	.14	.13	...
	Def.	...	...	*.12*	*.16*	*.21*	*.27*	*.33*	*.40*	*.48*	*.56*	*.65*	*.74*	*.85*	*.96*	*1.07*	*1.19*	*1.32*	...
18	59	...	...	44	37	33	29	26	24	22	20	19	17	16	15	15	14	13	...
	54.5	...	...	42	36	31	28	25	23	21	19	18	17	16	15	14	13	12	...
	49	...	...	39	34	30	26	24	21	20	18	17	16	15	14	13	12	12	...
	*	...	...	.39	.34	.29	.26	.24	.21	.20	.18	.17	.16	.15	.14	.13	.12	.12	...
	Def.	...	...	*.13*	*.18*	*.24*	*.30*	*.37*	*.44*	*.53*	*.62*	*.72*	*.83*	*.94*	*1.06*	*1.19*	*1.33*	*1.47*	
15	71.5	...	...	47	40	35	31	28	26	24	22	20	19	18	17	16	15	14	...
	54.5	...	...	36	31	27	24	22	20	18	17	15	14	14	13	12	11	11	...
	46	...	...	29	25	22	19	17	16	14	13	12	11	11	10	10	9	9	...
	41	...	...	27	23	20	18	16	15	14	12	12	11	10	10	9	9	8	...
	38.5	...	...	26	22	20	17	16	14	13	12	11	10	10	9	9	8	8	...
	*	...	...	.33	.28	.26	.22	.20	.18	.16	.15	.14	.13	.12	.12	.11	.10	.10	...
	Def.	...	...	*.16*	*.22*	*.28*	*.36*	*.44*	*.53*	*.64*	*.75*	*.87*	*.99*	*1.13*	*1.28*	*1.43*	*1.60*	*1.76*	...
12	36.5	...	24	20	17	15	13	12	11	10	9	9	8	7	7	...	...	...	...
	32	...	20	17	15	13	11	10	9	8	8	7	7	6	6	...	...	...	...
	28.5	...	19	16	14	12	11	10	9	8	7	7	6	6	6	...	...	...	...
	*	...	.31	.26	.22	.20	.17	.16	.14	.13	.12	.11	.11	.10	.09	...	...	...	...
	Def.	...	*.14*	*.20*	*.27*	*.35*	*.45*	*.55*	*.67*	*.79*	*.93*	*1.08*	*1.24*	*1.41*	*1.59*	...	...	...	...
10	28.5	...	14	12	10	9	8	7	7	6	6	5	5	...	...	...	...	...	...
	23.5	...	13	11	9	8	7	7	6	5	5	5	4	...	...	...	...	...	...
	*	...	.26	.22	.19	.16	.15	.13	.12	.11	.10	.09	.09	...	...	...	...	...	...
	Def.	...	*.17*	*.24*	*.32*	*.42*	*.54*	*.66*	*.80*	*.95*	*1.12*	*1.30*	*1.49*	...	...	...	...	...	...
9	24	14	11	9	8	7	6	5	5	5	4	4	4	...	...	...	...	...	...
	20.5	13	10	8	7	6	6	5	5	4	4	4	3	...	...	...	...	...	...
	*	.29	.24	.20	.17	.15	.13	.12	.11	.10	.09	.09	.08	...	...	...	...	...	...
	Def.	*.12*	*.18*	*.27*	*.36*	*.47*	*.60*	*.74*	*.89*	*1.06*	*1.24*	*1.44*	*1.66*	...	...	...	...	...	...
8	19.5	10	8	7	6	5.0	4.5	4.0	3.7	3.4	...	...	...	...	...	...	...	...	...
	17.5	10	8	6	5	4.8	4.2	3.8	3.5	3.2	...	...	...	...	...	...	...	...	...
	*	.26	.21	.17	.15	.13	.12	.11	.10	.09	...	...	...	...	...	...	...	...	...
	Def.	*.13*	*.21*	*.30*	*.41*	*.53*	*.67*	*.83*	*1.00*	*1.19*	...	...	...	...	...	...	...	...	...

The figures give the safe uniform load, in tons of 2000 lb., based on an extreme fiber stress of 16000 lb. per sq. in., or end reactions for safe uniform load in thousands of lb.

Figures for deflection in inches.

For loads concentrated at center, use one-half of figures given for allowable load, and four-fifths of deflections.

For figures to right of heavy lines, deflections are excessive for plastered ceilings.

Figures given apply only when beams are secured against lateral deformation.

* Increase of safe load in tons for each pound increase in weight of I-Beam.

TABLE 160.

Safe Loads, in Tons, and Deflections in Inches, Bethlehem Girder Beams.

Depth.	Weight.	Length of Span in Feet.																	
In.	Lb.	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40	42	44
30	200	......	......	......	......	181	163	148	136	125	116	108	102	96	90	86	81	77	74
	181	......	......	......	......	162	146	132	121	112	104	97	91	86	81	77	73	69	66
	*	......	......	......	......	.44	.39	.36	.33	.30	.28	.26	.25	.23	.22	.21	.20	.19	.18
	Def.	......	......	......	......	*.18*	*.22*	*.27*	*.32*	*.37*	*.43*	*.50*	*.57*	*.64*	*.71*	*.80*	*.88*	*.97*	*1.06*
28	180	......	......	......	......	154	138	126	115	106	99	92	86	81	77	73	69	66	63
	165	......	......	......	......	139	125	114	104	96	89	83	78	74	69	66	62	60	57
	*	......	......	......	......	.41	.37	.33	.31	.28	.26	.24	.23	.22	.20	.19	.18	.17	.17
	Def.	......	......	......	......	*.19*	*.24*	*.29*	*.34*	*.40*	*.46*	*.53*	*.61*	*.78*	*.77*	*.85*	*.95*	*1.04*	*1.14*
26	160	......	......	......	......	128	115	105	96	89	82	77	72	68	64	61	58	55	52
	151	......	......	......	......	117	106	96	88	81	76	70	66	62	59	56	53	50	48
	*	......	......	......	......	.38	.34	.31	.28	.26	.24	.23	.21	.20	.19	.18	.17	.16	.15
	Def.	......	......	......	......	*.21*	*.25*	*.31*	*.37*	*.43*	*.50*	*.57*	*.65*	*.74*	*.83*	*.92*	*1.02*	*1.12*	*1.23*
24	141	......	156	133	117	104	93	85	78	72	67	62	58	55	52	49	47	......	......
	121	......	134	115	100	89	80	73	67	62	57	53	50	47	45	42	40	......	......
	*	......	.52	.45	.39	.35	.31	.29	.26	.24	.22	.21	.20	.18	.17	.17	.16	......	......
	Def.	......	*.10*	*.14*	*.18*	*.22*	*.28*	*.33*	*.40*	*.47*	*.54*	*.62*	*.71*	*.80*	*.89*	*1.00*	*1.10*	......	......
20	142	......	130	112	98	87	78	71	65	60	56	52	49	46	43	41	39	......	......
	113	......	104	89	78	69	62	57	52	48	45	42	39	37	35	33	31	......	......
	*	......	.44	.37	.33	.29	.26	.24	.22	.20	.19	.17	.16	.15	.15	.14	.13	......	......
	Def.	......	*.12*	*.16*	*.21*	*.27*	*.33*	*.40*	*.48*	*.56*	*.65*	*.74*	*.85*	*.96*	*1.07*	*1.19*	*1.32*	......	......
18	93	......	79	67	59	52	47	43	39	36	34	31	29	28	26	25	24	......	......
	*	......	.39	.34	.29	.26	.24	.21	.20	.18	.17	.16	.15	.14	.13	.12	.12	......	......
	Def.	......	*.13*	*.18*	*.24*	*.30*	*.37*	*.44*	*.53*	*.62*	*.72*	*.83*	*.94*	*1.06*	*1.19*	*1.33*	*1.47*	......	......
15	141	113	94	81	71	63	57	51	47	44	40	38	35	33	......	......	......	......	......
	105	87	72	62	54	48	43	39	36	33	31	29	27	26	......	......	......	......	......
	74	63	52	45	39	35	31	29	26	24	22	21	20	18	......	......	......	......	......
	*	.39	.33	.28	.25	.22	.20	.18	.16	.15	.14	.13	.12	.12	......	......	......	......	......
	Def.	*.11*	*.16*	*.22*	*.28*	*.36*	*.44*	*.53*	*.64*	*.75*	*.87*	*.99*	*1.13*	*1.28*	......	......	......	......	......
12	70.5	48	40	34	30	27	24	22	20	18	17	16	15	14	......	......	......	......	......
	55.5	38	32	27	24	21	19	17	16	15	14	13	12	11	......	......	......	......	......
	*	.31	.26	.22	.20	.18	.16	.14	.13	.12	.11	.10	.10	.09	......	......	......	......	......
	Def.	*.14*	*.20*	*.27*	*.35*	*.45*	*.55*	*.67*	*.79*	*.93*	*1.08*	*1.24*	*1.41*	*1.59*	......	......	......	......	......
10	44.5	26	22	19	16	15	13	12	11	10	9	9	8	8	......	......	......	......	......
	*	.26	.22	.19	.16	.15	.13	.12	.11	.10	.09	.09	.08	.08	......	......	......	......	......
	Def.	*.17*	*.24*	*.32*	*.42*	*.54*	*.66*	*.80*	*.95*	*1.12*	*1.30*	*1.49*	*1.69*	*1.91*	......	......	......	......	......
9	38.5	20	17	14	13	11	10	9	8	8	7	7	......	......	......	......	......	......	......
	*	.23	.20	.17	.15	.13	.12	.11	.10	.09	.08	.07	......	......	......	......	......	......	......
	Def.	*.18*	*.27*	*.36*	*.47*	*.60*	*.74*	*.89*	*1.06*	*1.24*	*1.44*	*1.66*	......	......	......	......	......	......	......
8	33	15	13	11	10	8	8	7	6	......	......	......	......	......	......	......	......	......	......
	*	.21	.17	.15	.13	.12	.10	.09	.08	......	......	......	......	......	......	......	......	......	......
	Def.	*.21*	*.30*	*.41*	*.53*	*.67*	*.83*	*1.00*	*1.19*	......	......	......	......	......	......	......	......	......	......

The figures give the safe uniform load in tons, of 2000 lb., based on extreme fiber stress of 16000 lb. per sq. in., or end reactions for safe uniform load in thousands of pounds.

Figures for deflections are given in inches.

For load concentrated at center, use one-half of figures given for allowable load and four-fifths values given for deflection.

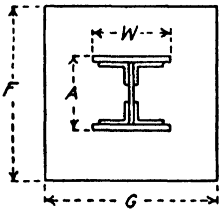
For figures at right of heavy zigzag lines deflections are considered excessive for plastered ceilings.

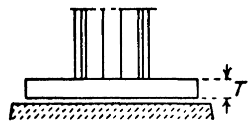
Figures given apply only when beams are secured against lateral deformation.

* Increase of safe load in tons for each pound increase in weight of Girder Beams.

TABLE 161.

STANDARD COLUMN FOOTINGS
ROLLED STEEL SLABS.
AMERICAN BRIDGE COMPANY.



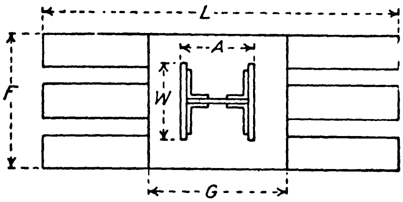


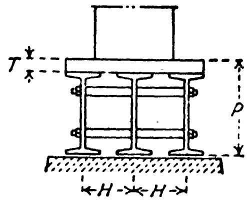
Load in 1000 Lb.	Min. Col.		Pressure 500 Pounds.						Pressure 600 Pounds.						Pressure 750 Pounds.					
			F'ng. No.	Slab.				Wt.	F'ng. No.	Slab.				Wt.	F'ng. No.	Slab.				Wt.
	A	W		T_1	T	F	G			T_1	T	F	G			T_1	T	F	G	
	In.	In.		In.	In.	In.	In.	Lb.		In.	In.	In.	In.	Lb.		In.	In.	In.	In.	Lb.
50	10½	8½	50B	1.2	1¼	14	10	50	60B	1.2	1¼	14	10	50	70B	1.2	1¼	14	10	50
75			50C	1.3	1¼	14	13	64	60C	1.3	1¼	14	13	64	70C	1.3	1¼	14	13	64
100			501	1.3	1¼	15	14	74	601	1.3	1½	16	11	75	701	1.3	1½	16	11	75
125			501A	1.4	1½	16	16	109	601A	1.4	1½	16	13	88	701A	1.5	1½	16	11	75
150	10½	10½	501B	1.4	1½	19	16	129	601B	1.3	1½	16	16	109	701B	1.5	1½	16	13	88
175			501C	1.5	2	20	18	204	601C	1.4	1½	18	16	122	701C	1.5	1½	16	15	102
200			502	1.8	2	20	20	227	602	1.7	2	20	17	193	702	1.4	1½	17	16	116
250			502B	2.0	2½	24	21	357	602B	2.0	2	21	20	238	702B	1.9	2	20	17	193
300	10½	12½	503	2.3	2½	25	24	425	603	2.4	2½	24	21	357	703	1.9	2	20	20	227
350			503B	2.8	3	28	25	595	603B	2.5	2½	25	24	425	703B	2.6	2½	24	20	340
400			504	2.9	3	29	28	690	604	3.0	3	28	24	570	704	2.5	2½	24	24	408
450			504B	3.4	3½	32	28	888	604B	3.0	3	28	27	643	704B	2.8	3	25	24	510
500	12½	14	505	3.1	3½	32	31	984	605	3.1	3	30	28	714	705	3.0	3	28	24	570
550			505B	3.5	3½	35	32	1 110	605B	3.4	3½	32	29	920	705B	3.0	3	28	27	643
600			506	3.7	4	36	33	1 347	606	3.4	3½	32	31	984	706	3.6	3½	32	28	888
650			506B	3.8	4	36	36	1 470	606B	4.1	4	36	30	1 225	706B	3.6	3½	32	30	953
700	14½	14	507	3.9	4	39	36	1 592	607	3.7	4	36	33	1 347	707	3.0	3½	32	29	920
800			508	4.4	4½	40	40	2 040	608	4.4	4½	40	34	1 735	708	4.1	4	35	30	1 190
900			509	4.6	5	44	41	2 557	609	4.5	4½	40	38	1 940	709	4.2	4½	36	34	1 562
1 000			5010	5.0	5	46	44	2 870	6010	5.1	5	44	38	2 370	7010	4.9	5	40	34	1 926
1 100	16½	16	....	....	....	....	....	....	6 011	4.9	5	44	42	2 620	7011	4.6	4½	40	37	1 888
1 200			....	....	....	....	....	....	6 012	5.4	6	48	42	3 430	7012	5.1	5	44	40	2 495
1 300			....	....	....	....	....	....	....	....	....	....	....	....	7013	5.2	5	44	41	2 557
1 400			....	....	....	....	....	....	....	....	....	....	....	....	7014	5.5	6	45	42	3 215
1 500	18½	18	....	....	....	....	....	....	....	....	....	....	....	....	7015	5.7	6	48	42	3 430
1 600			....	....	....	....	....	....	....	....	....	....	....	....	7016	6.2	6	50	42	3 572

Note.—For allowable stresses and details of the design of "Standard Column Footings," see page 119, Part I. T_1 is theoretical thickness.

TABLE 161.—*Continued.*

Standard Column Footings.
Single Tier Grillage.
American Bridge Company.





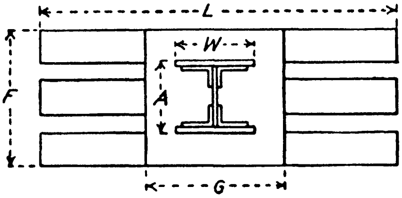
	Load in 1000 Lb.	Minimum Column.		Pressure 500 Pounds.									
				Footing Number.	P	Slab.				Beams.			Weight.
		A	W			T_1	T	F	G	H	Size.	L	
		In.	In.		Ft. In.	In.	In.	In.	In.	In.		Ft. In.	Lb.
Three Beam Grillage.	500	12½	14	515	1-2½	2.55	2½	22	24	8	3-12 I 35	4-0	813
	550			515B	1-3	2.75	3	22	24	8¼	3-12 I 35	4-3	914
	600			516	1-3	3.01	3	23	24	8½	3-12 I 45	4-6	1 096
	650			516B	1-6	3.00	3	23	24	8¼	3-15 I 50	4-11	1 227
	700	14½	14	517	1-6	2.96	3	22	24	8	3-15 I 65	5-4	1 525
	800			518	1-7	3.84	4	26	30	10	3-15 I 55	5-2	1 770
	900			519	1-10	4.06	4	27	30	10	3-18 I 65	5-9	2 072
	1 000			5110	2-0	4.03	4	26	30	9½	3-20 I 75	6-7	2 396
	1 100	16½	16	5111	2-0	4.06	4	28	30	10	3-20 I 85	6-9	2 708
	1 200			5112	2-0½	4.51	4½	30	34	11	3-20 I 85	6-11	3 102
	1 300			5113	2-4½	4.55	4½	29	34	10¾	3-24 I 100	7-7	3 567
	1 400			5114	2-5	5.00	5	31	34	11¼	3-24 I 120	7-8	4 291
	1 500	18½	18	5115	2-5½	5.53	5½	34	34	12¾	3-24 I 120	7-6	4 543
	1 600			5116	2-5½	5.55	5½	34	36	12¾	3-24 I 120	7-11	4 799
	1 700			5117	2-6	6.09	6	34	44	12¾	3-24 I 120	8-5	5 630
	1 800			5118	2-7	6.79	7	35	48	13½	3-24 I 120	8-6	6 452
Four Beam Grillage.	700	14½	14	527	1-1½	3.48	3½	28	32	7½	4-10 I 30	4-4	1 421
	800			528	1-4	4.01	4	30	28	7¾	4-12 I 40.8	4-8	1 739
	900			529	1-7	4.11	4	30	30	7¾	4-15 I 45	5-3	2 004
	1 000			5210	1-7½	4.33	4½	30	34	8	4-15 I 50	5-7	2 457
	1 100	16½	16	5211	1-7½	4.27	4½	31	34	8¼	4-15 I 60.8	6-0	2 845
	1 200			5212	1-10½	4.58	4½	32	34	8½	4-18 I 65	6-4	3 076
	1 300			5213	1-11	4.84	5	34	38	9	4-18 I 70	6-6	3 695
	1 400			5214	2-1½	5.50	5½	35	34	9½	4-20 I 75	6-8	3 902
	1 500	18½	18	5215	2-1½	5.43	5½	36	34	9½	4-20 I 85	7-0	4 334
	1 600			5216	2-2	6.00	6	38	34	10¼	4-20 I 90	7-0	4 767
	1 700			5217	2-6	5.90	6	37	34	9¾	4-24 I 100	7-10	5 318
	1 800			5218	2-6½	6.55	6½	40	35	10¾	4-24 I 100	7-7	5 663
	1 900	18½	18	5219	2-6½	6.42	6½	40	40	11	4-24 I 100	7-10	6 134
	2 000			5220	2-6½	6.57	6½	40	37	10½	4-24 I 120	8-5	6 835
	2 100			5221	2-6½	6.22	6½	40	44	10½	4-24 I 120	8-10	7 549
	2 200			5222	2-7	6.88	7	42	48	11¼	4-24 I 120	8-9	8 272
	2 300	20½	20	5223	2-7	6.70	7	44	48	12	4-24 I 120	8-8	8 426
	2 400			5224	2-7	7.08	7	48	50	12¾	4-24 I 120	8-8	9 003

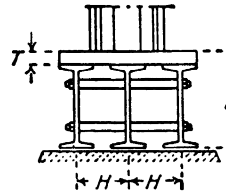
TABLE 161.—*Continued.*

Standard Column Footings.

Single Tier Grillage.

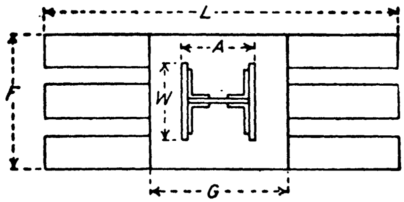
American Bridge Company.

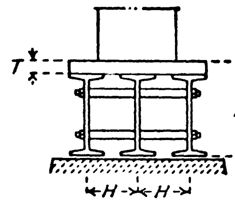




	Load in 1000 Lb.	Minimum Column.		Pressure 500 Pounds.									
				Footing Number.	P	Slab.				Beams.			Weight.
		A	W			T_1	T	F	G	H	Size.	L	
		In.	In.		Ft. In.	In.	In.	In.	In.	In.		Ft. In.	Lb.
Three Beam Grillage.	500	12½	14	535	1-3	2.70	3	22	24	8	3-12 I 35	4-0	887
	550			535B	1-3	2.82	3	22	24	8¼	3-12 I 35	4-3	914
	600			536	1-3	2.89	3	23	24	8½	3-12 I 45	4-6	1 096
	650			536B	1-6	3.01	3	23	24	8¼	3-15 I 50	4-11	1 227
	700	14½	14	537	1-6	2.89	3	24	22	9	3-15 I 55	4-11	1 281
	800			538	1-9	2.88	3	24	22	8¾	3-18 I 65	5-8	1 585
	900			539	1-10	4.02	4	30	25	10½	3-18 I 65	5-6	1 958
	1 000			5310	2-0	3.97	4	30	28	10¼	3-20 I 75	6-3	2 392
	1 100	16½	16	5311	1-10	4.02	4	30	29	11¼	3-18 I 85	6-3	2 618
	1 200			5312	2-4	3.94	4	30	28	10¾	3-24 I 100	7-0	3 087
	1 300			5313	2-4	4.00	4	30	27	10½	3-24 I 120	7-6	3 653
	1 400			5314	2-4½	4.46	4½	34	30	11¼	3-24 I 120	7-8	4 095
	1 500	18½	18	5315	2-4½	4.46	4½	34	32	12	3-24 I 120	7-10	4 246
	1 600			5316	2-5½	5.35	5½	34	36	12¾	3-24 I 120	7-11	4 799
	1 700			5317	2-6½	6.12	6½	35	40	13½	3-24 I 120	8-1	5 548
	1 800			5318	2-8	7.53	8	35	48	13½	3-24 I 120	8-6	6 928
Four Beam Grillage	700	14½	14	547	1-2	3.59	4	30	28	8¼	4-10 I 30	4-0	1 446
	800			548	1-4	3.63	4	30	27	7¾	4-12 I 40.8	4-8	1 706
	900			549	1-7	4.07	4	30	26	8	4-15 I 50	5-1	1 941
	1 000			5410	1-7½	4.54	4½	34	30	9	4-15 I 50	5-2	2 379
	1 100	16½	16	5411	1-7½	4.41	4½	34	30	9	4-15 I 60.8	5-7	2 705
	1 200			5412	1-10½	4.54	4½	34	31	9	4-18 I 65	6-0	2 949
	1 300			5413	1-11	4.78	5	35	34	9½	4-18 I 70	6-3	3 483
	1 400			5414	2-1	4.99	5	35	34	9½	4-20 I 75	6-8	3 733
	1 500	18½	18	5415	2-1	4.75	5	36	34	9½	4-20 I 85	7-0	4 161
	1 600			5416	2-1½	5.39	5½	38	34	10¼	4-20 I 90	7-0	4 584
	1 700			5417	2-5½	5.22	5½	37	34	9¾	4-24 I 100	7-10	5 141
	1 800			5418	2-6	6.02	6	40	34	10¾	4-24 I 100	7-7	5 398
	1 900	18½	18	5419	2-7	6.94	7	44	34	12	4-24 I 100	7-4	5 959
	2 000			5420	2-6½	6.56	6½	44	40	12	4-24 I 100	7-8	6 369
	2 100			5421	2-6½	6.28	6½	41	40	11	4-24 I 120	8-6	7 172
	2 200			5422	2-7	7.04	7	48	46	13¼	4-24 I 120	7-9	8 164
	2 300	20½	20	5423	2-7	7.06	7	48	42	13	4-24 I 120	8-2	8 002
	2 400			5424	2-7	6.95	7	48	47	13½	4-24 I 120	8-4	8 560

TABLE 161.—*Continued.*
Standard Column Footings.
Single Tier Grillage.
American Bridge Company.

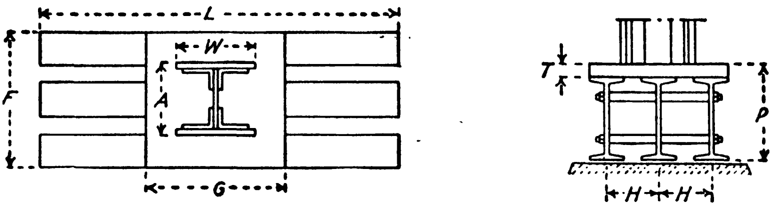




	Load in 1000 Lb.	Minimum Column.		Pressure 600 Pounds.									
				Footing Number.	P	Slab.				Beams.			Weight.
		A	W			T_1	T	F	G	H	Size.	L	
		In.	In.		Ft. In.	In.	In.	In.	In.	In.		Ft. In.	Lb.
Three Beam Grillage.	500	12½	14	615	1-0½	2.55	2½	22	24	8	3-10 I 30	3-4	684
	550			615B	1-1	2.76	3	22	24	8¼	3-10 I 30	3-7	780
	600			616	1-3	2.88	3	22	24	8¼	3-12 I 40.8	3-10	937
	650			616B	1-3	3.00	3	22	24	8¼	3-12 I 45	4-2	1 030
	700	14½	14	617	1-6	2.96	3	22	24	8	3-15 I 50	4-6	1 142
	800			618	1-7	3.84	4	26	30	10	3-15 I 50	4-4	1 557
	900			619	1-7	4.06	4	26	30	10	3-15 I 55	4-11	1 717
	1 000			6110	1-10	4.03	4	26	30	9½	3-18 I 65	5-6	1 989
	1 100	16½	16	6111	1-10½	4.31	4½	29	34	11	3-18 I 65	5-5	2 351
	1 200			6112	2-0½	4.51	4½	29	34	11	3-20 I 75	5-11	2 626
	1 300			6113	2-0½	4.55	4½	29	34	10¾	3-20 I 95	6-4	3 097
	1 400			6114	2-1	5.00	5	30	34	11¼	3-20 I 95	6-6	3 334
	1 500	18½	18	6115	2-5½	5.23	5½	34	38	12¾	3-24 I 100	6-5	3 980
	1 600			6116	2-5½	5.43	5½	34	41	12¾	3-24 I 100	6-10	4 265
	1 700			6117	2-5½	5.51	5½	34	39	12¾	3-24 I 120	7-0	4 627
	1 800			6118	2-6	5.93	6	34	42	12¾	3-24 I 120	7-5	5 141
	1 900	18½	18	6119	2-7	6.61	7	34	45	12¾	3-24 I 120	7-10	5 897
	2 000			6120	2-7	7.19	7	35	48	13¼	3-24 I 120	7-11	6 227
Four Beam Grillage.	1 100	14½	14	6211	1-7	4.05	4	30	33	7¾	4-15 I 50	5-3	2 211
	1 200			6212	1-7½	4.46	4½	31	34	8¼	4-15 I 60.8	5-5	2 702
	1 300			6213	1-8	5.04	5	34	34	9	4-15 I 60.8	5-6	3 021
	1 400			6214	1-11½	5.50	5½	35	34	9½	4-18 I 65	5-7	3 352
	1 500	18½	18	6215	1-11½	5.54	5½	36	34	9¾	4-18 I 70	5-10	3 588
	1 600			6216	2-1½	5.58	5½	36	34	9½	4-20 I 85	6-3	4 079
	1 700			6217	2-0	6.03	6	37	34	10	4-18 I 85	6-4	4 342
	1 800			6218	2-0½	6.38	6½	40	37	10½	4-18 I 85	6-4	4 933
	1 900	18½	18	6219	2-2½	6.46	6½	40	38	10¾	4-20 I 90	6-8	5 253
	2 000			6220	2-2½	6.31	6½	40	42	10¾	4-20 I 95	7-0	5 807
	2 100			6221	2-6½	6.46	6½	40	42	10¼	4-24 I 100	7-5	6 113
	2 200			6222	2-6½	6.55	6½	40	45	11	4-24 I 100	7-7	6 402
	2 300	20½	20	6223	2-6½	6.27	6½	40	42	10½	4-24 I 120	8-2	7 083
	2 400			6224	2-7	6.92	7	40	48	10½	4-24 I 120	8-5	7 917
	2 500			6225	2-7	6.80	7	43	48	11½	4-24 I 120	8-2	8 089
	2 600			6226	2-7	7.12	7	44	48	12	4-24 I 120	8-2	8 188

TABLE 161.—*Continued.*

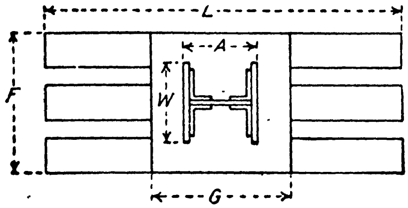
STANDARD COLUMN FOOTINGS.
SINGLE TIER GRILLAGE.
AMERICAN BRIDGE COMPANY.

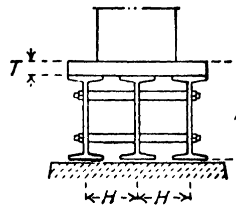


	Load in 1000 Lb.	Minimum Column. A	Minimum Column. W	Footing Number.	P	Slab. T_1	Slab. T	Slab. F	Slab. G	Beams. H	Beams. Size.	Beams. L	Weight.
		In.	In.		Ft. In.	In.	In.	In.	In.	In.		Ft. In.	Lb.
													Pressure 600 Pounds.
Three Beam Grillage.	500	12½	14	635	1-1	2.70	3	22	24	8¼	3-10 I 30	3-3	750
	550			635B	1-1	2.82	3	22	24	8½	3-10 I 30	3-6	773
	600			636	1-3	2.95	3	22	24	8¼	3-12 I 40.8	3-10	937
	650			636B	1-3	3.01	3	23	24	8¼	3-12 I 45	4-2	1 052
	700	14½	14	637	1-3	3.01	3	24	23	9¼	3-12 I 45	4-1	1 042
	800			638	1-6	3.07	3	24	23	9	3-15 I 55	4-8	1 259
	900			639	1-9	3.08	3	24	22	8¾	3-18 I 70	5-3	1 581
	1 000			6310	1-10	4.00	4	30	26	10¼	3-18 I 70	5-3	2 022
	1 100	16½	16	6311	1-10	3.93	4	30	28	11	3-18 I 70	5-5	2 128
	1 200			6312	1-10	3.94	4	30	28	10¾	3-18 I 85	5-10	2 475
	1 300			6313	2-0	4.00	4	30	27	10½	3-20 I 100	6-5	2 879
	1 400			6314	2-0½	4.55	4½	34	31	11½	3-20 I 100	6-5	3 307
	1 500	18½	18	6315	2-4½	4.50	4½	34	32	12¼	3-24 I 120	6-5	3 738
	1 600			6316	2-5½	5.35	5½	34	36	12¾	3-24 I 120	6-8	4 346
	1 700			6317	2-6	6.03	6	34	39	12¾	3-24 I 120	7-1	4 847
	1 800			6318	2-7	6.72	7	34	42	12¾	3-24 I 120	7-6	5 575
	1 900	18½	18	6319	2-8	7.85	8	34	48	12¾	3-24 I 120	7-10	6 563
	2 000			6320	2-8	7.92	8	35	48	13½	3-24 I 120	7-11	6 703
Four Beam Grillage.	1 100	16½	16	6411	1-7	4.05	4	31	30	8¼	4-15 I 50	5-0	2 082
	1 200			6412	1-7½	4.52	4½	34	29	8¾	4-15 I 60.8	5-2	2 558
	1 300			6413	1-8	4.78	5	34	32	9¼	4-15 I 60.8	5-4	2 884
	1 400			6414	1-11	4.89	5	34	33	9¼	4-18 I 65	5-9	3 131
	1 500	18½	18	6415	2-1	4.75	5	35	34	9½	4-20 I 75	6-0	3 533
	1 600			6416	2-1	4.91	5	36	34	9½	4-20 I 85	6-3	3 907
	1 700			6417	2-1	5.05	5	36	34	9½	4-20 I 90	6-8	4 181
	1 800			6418	2-1½	5.55	5½	38	34	10	4-20 I 95	6-8	4 596
	1 900	18½	18	6419	2-5½	5.52	5½	37	34	9¾	4-24 I 100	7-3	4 908
	2 000			6420	2-6½	6.03	6½	40	38	10¾	4-24 I 100	7-1	5 685
	2 100			6421	2-6½	6.35	6½	40	40	10¾	4-24 I 100	7-4	5 933
	2 200			6422	2-6½	6.50	6½	40	40	10½	4-24 I 120	7-10	6 754
	2 300	20½	20	6423	2-6½	6.59	6½	40	42	10½	4-24 I 120	8-2	7 066
	2 400			6424	2-7	6.94	7	48	47	12¾	4-24 I 100	7-3	7 436
	2 500			6425	2-7	7.00	7	48	45	12¾	4-24 I 120	7-6	7 946
	2 600			6426	2-7	7.06	7	48	46	12¾	4-24 I 120	7-9	8 162

TABLE 161.—*Continued.*

STANDARD COLUMN FOOTINGS. SINGLE TIER GRILLAGE. AMERICAN BRIDGE COMPANY.

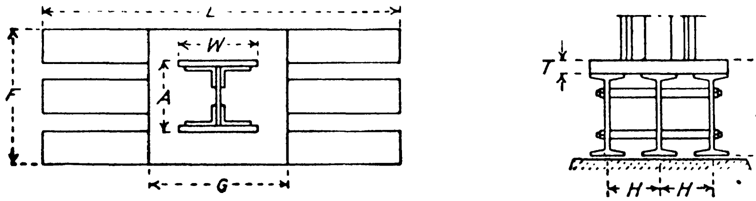




	Load in 1000 Lb.	Minimum Column.		Pressure 750 Pounds.									
		A	W	Footing Number.	P	Slab. T_1	Slab. T	Slab. F	Slab. G	Beams. H	Beams. Size.	Beams. L	Weight.
		In.	In.		Ft. In.	In.	In.	In.	In.	In.		Ft. In	Lb.
Three Beam Grillage.	700	14½	14	717	1-3	2.96	3	22	24	8	3-12 I 45	3-7	949
	800			718	1-3	3.04	3	24	26	8	3-12 I 50	4-2	1 174
	900			719	1-4	3.92	4	24	30	9¼	3-12 I 45	4-2	1 402
	1 000			7110	1-7	4.05	4	25	30	9½	3-15 I 55	4-6	1 614
	1 100	16½	16	7111	1-7	4.06	4	27	30	10	3-15 I 65	4-9	1 866
	1 200			7112	1-10	4.09	4	26	30	9¾	3-18 I 70	5-3	2 020
	1 300			7113	1-11	4.71	5	27	34	10¼	3-18 I 70	5-5	2 472
	1 400			7114	1-11	4.73	5	29	34	10½	3-18 I 90	5-7	2 941
	1 500	18½	18	7115	1-11	4.87	5	31	34	11½	3-18 I 90	5-6	3 017
	1 600			7116	2-1	5.03	5	31	34	11½	3-20 I 95	5-11	3 219
	1 700			7117	2-1½	5.46	5½	34	38	12½	3-20 I 95	5-11	3 740
	1 800			7118	2-1½	5.59	5½	34	40	12½	3-20 I 100	6-3	4 034
	1 900	18½	18	7119	2-3	6.60	7	34	45	12½	3-20 I 100	6-6	5 026
	2 000			7120	2-8	7.75	8	30	48	11	3-24 I 120	7-5	5 972
Four Beam Grillage.	1 100	16½	16	7211	1-2½	4.40	4½	31	34	8½	4-10 I 35	4-0	1 918
	1 200			7212	1-4½	4.45	4½	31	34	8¼	4-12 I 45	4-5	2 168
	1 300			7213	1-5	5.08	5	33	34	9	4-12 I 50	4-5	2 503
	1 400			7214	1-8	5.09	5	32	34	8¾	4-15 I 55	4-10	2 633
	1 500	18½	18	7215	1-8½	5.57	5½	36	34	9¾	4-15 I 55	4-9	2 984
	1 600			7216	1-8½	5.57	5½	35	34	9½	4-15 I 65	5-1	3 222
	1 700			7217	2-0	5.75	6	35	34	9½	4-18 I 70	5-5	3 587
	1 800			7218	2-0	5.92	6	36	34	9½	4-18 I 85	5-8	4 054
	1 900	18½	18	7219	2-0	6.08	6	36	34	9½	4-18 I 90	5-11	4 257
	2 000			7220	2-1	6.86	7	39	34	10½	4-18 I 90	5-9	4 762
	2 100			7221	2-3	6.90	7	39	34	10¼	4-20 I 95	6-3	5 055
	2 200			7222	2-2½	6.50	6½	38	40	10¼	4-20 I 95	6-5	5 288
	2 300	20½	20	7223	2-2½	6.53	6½	40	41	10¾	4-20 I 95	6-4	5 479
	2 400			7224	2-2½	6.57	6½	40	46	10¾	4-20 I 100	6-9	6 143
	2 500			7225	2-3	6.87	7	42	48	11½	4-20 I 100	6-8	6 720
	2 600			7226	2-3	7.10	7	44	48	12	4-20 I 100	6-8	6 814

TABLE 161.—*Continued.*

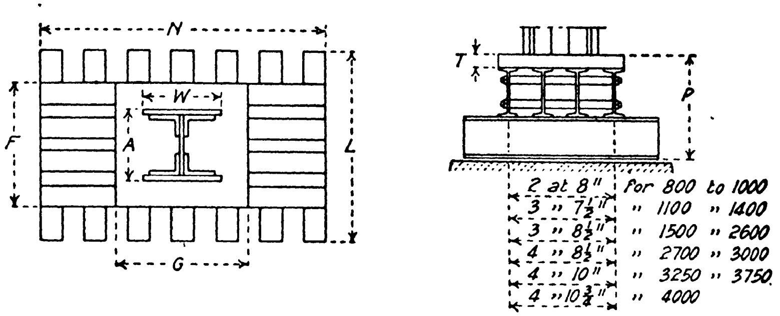
STANDARD COLUMN FOOTINGS.
SINGLE TIER GRILLAGE.
AMERICAN BRIDGE COMPANY.



	Load in 1000 Lb.	Minimum Column.		Pressure 750 Pounds.									
				Footing Number.	P	Slab.				Beams.			Weight.
		A	W			T_1	T	F	G	H	Size.	L	
		In.	In.		Ft. In.	In.	In.	In.	In.	In.		Ft. In.	Lb.
Three Beam Grillage.	700	14½	14	737	1-3	2.89	3	24	22	9	3-12 I 45	3-4	919
	800			738	1-3	3.07	3	24	23	8¾	3-12 I 50	3-11	1 077
	900			739	1-6	3.05	3	24	22	8½	3-15 I 65	4-4	1 312
	1 000			7310	1-6	2.99	3	24	21	8¾	3-15 I 75	4-10	1 536
	1 100	16½	16	7311	1-7	3.75	4	30	26	10½	3-18 I 70	4-6	1 852
	1 200			7312	1-10	3.94	4	30	28	10½	3-18 I 70	4-11	2 008
	1 300			7313	1-10	4.00	4	30	27	10½	3-18 I 90	5-2	2 349
	1 400			7314	2-0	4.03	4	30	27	10¼	3-20 I 100	5-8	2 653
	1 500	18½	18	7315	2-0	3.94	4	30	28	11	3-20 I 100	5-9	2 714
	1 600			7316	2-0½	4.60	4½	34	30	12	3-20 I 100	5-9	3 065
	1 700			7317	2-1	4.96	5	34	33	12½	3-20 I 100	5-11	3 406
	1 800			7318	2-2	5.85	6	34	37	12¾	3-20 I 100	6-1	4 007
	1 900	18½	18	7319	2-4	7.85	8	34	48	12½	3-20 I 95	6-6	5 592
	2 000			7320	2-4	8.06	8	34	48	12¾	3-24 I 120	6-8	6 143
Four Beam Grillage.	1 100	16½	16	7411	1-4	4.04	4	31	30	8¼	4-12 I 45	4-1	1 818
	1 200			7412	1-4½	4.53	4½	34	31	9	4-12 I 45	4-2	2 124
	1 300			7413	1-5	4.94	5	34	32	9½	4-12 I 45	4-3	2 338
	1 400			7414	1-8	5.05	5	34	31	9¼	4-15 I 55	4-8	2 551
	1 500	18½	18	7415	1-8	4.90	5	36	34	9¾	4-15 I 55	4-9	2 812
	1 600			7416	1-8	5.07	5	36	34	9¾	4-15 I 65	5-0	3 066
	1 700			7417	1-11	5.05	5	36	34	9½	4-18 I 70	5-5	3 297
	1 800			7418	1-11½	5.20	5½	36	34	9½	4-18 I 85	5-8	3 880
	1 900	18½	18	7419	1-11½	5.35	5½	36	34	9½	4-18 I 90	5-11	4 084
	2 000			7420	2-1½	5.48	5½	36	34	9½	4-20 I 95	6-2	4 329
	2 100			7421	2-2	6.00	6	37	34	10	4-20 I 100	6-3	4 690
	2 200			7422	2-2½	6.49	6½	40	36	10¾	4-20 I 100	6-2	5 171
	2 300	20½	20	7423	2-2½	6.41	6½	40	41	10¾	4-20 I 100	6-6	5 673
	2 400			7424	2-2½	6.50	6½	42	40	11½	4-20 I 100	6-4	5 683
	2 500			7425	2-1	6.86	7	48	45	12½	4-18 I 90	6-2	6 565
	2 600			7426	2-3	6.99	7	48	45	12½	4-20 I 100	6-6	6 945

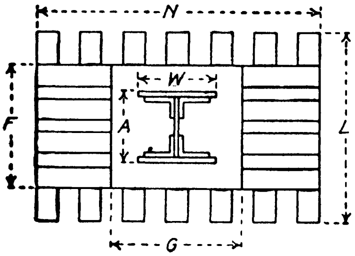
TABLE 161.—*Continued.*

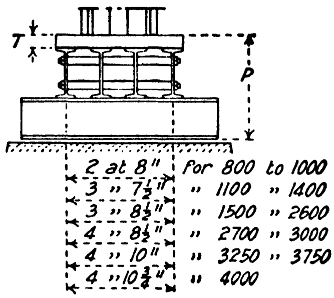
Standard Column Footings.
Double Tier Grillage.
American Bridge Company.



Load in 1000 Lb.	Min. Col.		Pressure 500 Pounds.										
	A	W	Footing Number.	P	Slab. T_1	T	F	G	Upper Beams. Size.	N	Lower Beams. Size.	L	Weight.
	In.	In.		Ft. In.	In.	In.	In.	In.		Ft. In.		Ft. In.	Lb.
800	14½	14	558	1-11	2.88	3	24	17	3-12 I 55	3-6	7-8 I 20.5	3-2	1 412
900			559	2-1	2.83	3	24	20	3-12 I 55	3-6	6-10 I 30	3-7	1 662
1 000			5510	2-3	2.99	3	24	21	3-12 I 55	3-6	6-12 I 35	4-0	1 879
1 100	16½	16	5511	2-5	3.99	4	30	22	4-15 I 55	4-4	7-10 I 30	3-7	2 496
1 200			5512	2-5	3.99	4	30	24	4-15 I 55	4-4	7-10 I 30	3-10	2 616
1 300			5513	2-7	3.99	4	30	26	4-15 I 55	4-4	7-12 I 35	4-2	2 903
1 400			5514	2-10	4.07	4	30	27	4-18 I 70	4-4	7-12 I 40.8	4-6	3 466
1 500	18½	18	5515	2-10½	4.31	4½	34	30	4-18 I 70	4-10	7-12 I 35	4-4	3 767
1 600			5516	2-10½	4.44	4½	34	30	4-18 I 70	4-10	7-12 I 40.8	4-7	4 016
1 700			5517	2-11	4.57	5	34	31	4-18 I 70	4-10	8-12 I 40.8	4-11	4 509
1 800			5518	3-2½	5.28	5½	34	34	4-18 I 70	4-10	7-15 I 50	5-2	5 016
1 900	18½	18	5519	3-4	5.11	5	34	27	4-20 I 100	5-4	7-15 I 50	4-11	5 231
2 000			5520	3-4	5.07	5	34	29	4-20 I 100	5-4	7-15 I 55	5-2	5 596
2 100			5521	3-7	5.10	5	34	30	4-20 I 100	5-4	7-18 I 60	5-6	5 965
2 200			5522	3-7½	5.42	5½	34	32	4-20 I 100	5-4	7-18 I 60	5-9	6 320
2 300	20½	20	5523	3-7½	5.29	5½	34	33	4-20 I 100	5-10	7-18 I 65	5-6	6 659
2 400			5524	3-8	5.84	6	34	35	4-20 I 100	5-10	7-18 I 65	5-9	7 049
2 500			5525	3-11	6.43	7	34	37	4-20 I 100	5-10	7-20 I 75	5-11	8 010
2 600			5526	3-11	6.98	7	34	39	4-20 I 100	5-10	7-20 I 75	6-3	8 319
2 700	22½	22	5527	3-8	6.04	6	42	35	5-20 I 100	6-2	8-18 I 60	6-1	8 596
2 800			5528	3-8	6.07	6	42	36	5-20 I 100	6-2	8-18 I 60	6-4	8 786
2 900			5529	3-10	6.09	6	42	37	5-20 I 100	6-2	7-20 I 75	6-7	9 276
3 000			5530	3-10	6.12	6	42	38	5-20 I 100	6-2	8-20 I 75	6-9	9 941
3 250	22½	22	5532	3-11	7.18	7	48	44	5-20 I 100	6-2	7-20 I 90	7-4	11 955
3 500			5535	4-4	7.45	8	48	44	5-20 I 100	6-2	7-24 I 100	7-11	13 475
3 750	24½	24	5537	4-4	7.76	8	48	47	5-20 I 100	6-3	8-24 I 100	8-4	15 975
4 000			5540	4-4	7.98	8	51	48	5-20 I 100	6-3	7-24 I 120	8-11	16 239

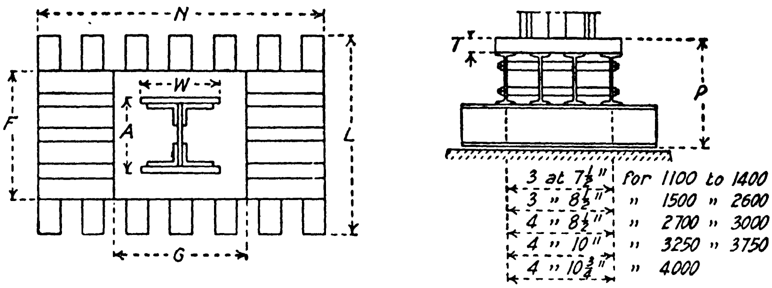
TABLE 161.—*Continued.*
STANDARD COLUMN FOOTINGS.
DOUBLE TIER GRILLAGE.
AMERICAN BRIDGE COMPANY.





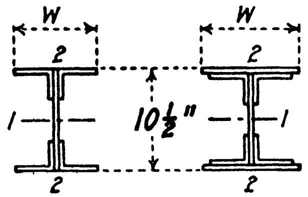
Load in 1000 Lb.	Min. Col.		Pressure 600 Pounds.										
			Footing Number.	*P*	Slab.				Upper Beams.		Lower Beams.		Weight.
	A	*W*			T_1	*T*	*F*	*G*	Size.	*N*	Size.	*L*	
	In.	In.		Ft. In.	In.	In.	In.	In.		Ft. In.		Ft. In.	Lb.
800	1 ½	14	658	1-11	2.88	3	24	17	3-12 I 55	3-6	7-8 I 20.5	2-8	1 339
900			659	1-11	2.83	3	24	20	3-12 I 55	3-6	7-8 I 20.5	3-0	1 449
1 000			6510	2-1	2.99	3	24	21	3-12 I 55	3-6	6-10 I 30	3-4	1 637
1 100	16½	16	6511	2-3	3.99	4	30	22	4-15 I 55	4-4	8-8 I 20.5	3-0	2 236
1 200			6512	2-3	3.99	4	30	24	4-15 I 55	4-4	8-8 I 20.5	3-3	2 346
1 300			6513	2-5	3.99	4	30	26	4-15 I 55	4-4	7-10 I 30	3-6	2 616
1 400			6514	2-8	4.07	4	30	27	4-18 I 70	4-4	7-10 I 30	3-9	2 969
1 500	18½	18	6515	2-6½	4.31	4½	34	30	4-18 I 70	4-10	9-8 I 20.5	3-7	3 367
1 600			6516	2-8½	4.44	4½	34	30	4-18 I 70	4-10	8-10 I 30	3-10	3 625
1 700			6517	2-11	4.57	5	34	31	4-18 I 70	4-10	7-12 I 40.8	4-1	4 063
1 800			6518	2-11½	5.28	5½	34	34	4-18 I 70	4-10	7-12 I 40.8	4-4	4 443
1 900	18½	18	6519	3-1	5.11	5	34	27	4-20 I 100	5-4	8-12 I 35	4-2	4 678
2 000			6520	3-1	5.07	5	34	29	4-20 I 100	5-4	8-12 I 40.8	4-4	5 019
2 100			6521	3-4	5.10	5	34	30	4-20 I 100	5-4	7-15 I 50	4-7	5 287
2 200			6522	3-4½	5.42	5½	34	32	4-20 I 100	5-4	7-15 I 50	4-9	5 565
2 300	20½	20	6523	3-4½	5.29	5½	34	33	4-20 I 100	5-10	8-15 I 50	4-7	5 989
2 400			6524	3-5	5.84	6	34	35	4-20 I 100	5-10	8-15 I 50	4-9	6 333
2 500			6525	3-6	6.43	7	34	37	4-20 I 100	5-10	8-15 I 50	5-0	6 903
2 600			6526	3-9	6.98	7	34	39	4-20 I 100	5-10	7-18 I 65	5-2	7 390
2 700	22½	22	6527	3-5	6.04	6	42	35	5-20 I 100	6-2	8-15 I 50	5-1	7 709
2 800			6528	3-5	6.07	6	42	36	5-20 I 100	6-2	8-15 I 50	5-3	7 848
2 900			6529	3-8	6.09	6	42	37	5-20 I 100	6-2	8-18 I 60	5-6	8 460
3 000			6530	3-8	6.12	6	42	38	5-20 I 100	6-2	8-18 I 60	5-8	8 612
3 250	22½	22	6532	3-9	7.18	7	48	44	5-20 I 100	6-2	8-18 I 60	6-2	10 295
3 500			6535	4-0	7.45	8	48	44	5-20 I 100	6-2	7-20 I 75	6-7	11 395
3 750	24½	24	6537	4-0	7.76	8	48	47	5-20 I 100	6-3	7-20 I 90	7-0	12 715
4 000			6540	4-0	7.98	8	51	48	5-20 I 100	6-3	8-20 I 90	7-5	14 079

TABLE 161.—*Continued.*
Standard Column Footings.
Double Tier Grillage.
American Bridge Company.



Load in 1000 Lb.	Min. Col.		Pressure 750 Pounds.										
	A	W	Footing Number.	P	Slab. T₁	Slab. T	Slab. F	Slab. G	Upper Beams. Size.	Upper Beams. N	Lower Beams. Size.	Lower Beams. L	Weight.
	In.	In.		Ft. In.	In.	In.	In.	In.		Ft. In.		Ft. In.	Lb.
1 100	16½	16	7511	2-0	4.00	4	30	22	4-12 I 50	3-6	7- 8 I 20.5	2-11	1 908
1 200			7512	2-0	3.99	4	30	24	4-12 I 50	3-6	7- 8 I 20.5	3-2	2 012
1 300			7513	2-2	3.99	4	30	26	4-12 I 50	3-6	6-10 I 30	3-5	2 241
1 400			7514	2-2	4.04	4	30	27	4-12 I 50	3-6	6-10 I 35	3-8	2 429
1 500	18½	18	7515	2-5	3.96	4	30	22	4-15 I 75	4-0	7-10 I 30	3-6	2 732
1 600			7516	2-5	3.99	4	30	23	4-15 I 75	4-0	7-10 I 35	3-8	2 929
1 700			7517	2-7	4.03	4	30	24	4-15 I 75	4-0	6-12 I 45	3-11	3 123
1 800			7518	2-7	4.08	4	30	25	4-15 I 75	4-0	6-12 I 50	4-2	3 350
1 900	18½	18	7519	2-8	5.11	5	34	27	4-15 I 75	4-4	6-12 I 45	4-1	3 756
2 000			7520	2-8	5.07	5	34	29	4-15 I 75	4-4	6-12 I 45	4-3	3 899
2 100			7521	2-11	5.09	5	34	30	4-15 I 75	4-4	6-15 I 55	4-6	4 283
2 200			7522	2-11½	5.42	5½	34	32	4-15 I 75	4-4	6-15 I 55	4-8	4 889
2 300	20½	20	7523	3-4½	5.29	5½	34	33	4-20 I 100	4-10	7-15 I 55	4-5	5 445
2 400			7524	3-5	5.84	6	34	35	4-20 I 100	4-10	7-15 I 55	4-7	5 785
2 500			7525	3-6	6.43	7	34	37	4-20 I 100	4-10	7-15 I 65	4-10	6 687
2 600			7526	3-6	6.98	7	34	39	4-20 I 100	4-10	7-15 I 65	5-0	6 899
2 700	22½	22	7527	3-5	6.04	6	42	35	5-20 I 100	5-2	7-15 I 50	4-10	6 868
2 800			7528	3-5	6.07	6	42	36	5-20 I 100	5-2	7-15 I 55	5-0	7 173
2 900			7529	3-5	6.09	6	42	37	5-20 I 100	5-2	8-15 I 50	5-2	7 386
3 000			7530	3-5	6.12	6	42	38	5-20 I 100	5-2	8-15 I 55	5-5	7 776
3 250	22½	22	7532	3-6	7.18	7	48	44	5-20 I 100	5-8	8-15 I 50	5-4	9 215
3 500			7535	3-7	7.45	8	48	44	5-20 I 100	5-8	8-15 I 60.8	5-9	10 485
3 750	24½	24	7537	3-7	7.76	8	48	47	5-20 I 100	6-0	8-15 I 65	5-10	11 215
4 000			7540	3-10	7.98	8	51	48	5-20 I 100	6-0	8-18 I 65	6-2	11 829

TABLE 162.
Constant Dimension Columns.
American Bridge Company.



When specifying these columns, give either the Section Number and weight per foot, thus: 10 AB 117, or the Index Number thus: INDEX 1015.

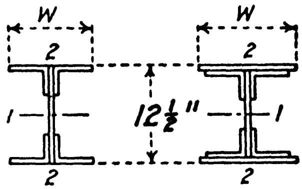
Rivets ¾"

Section Number.	Weight per Ft.	Index Number.	Width W	Material. Web Plate.	Material. Four Angles.	Material. Two Cover Plates.	Area.	Properties. Axis 1-1. Sec. Mod. S	Properties. Axis 1-1. Rad. Gyr. r	Properties. Axis 2-2. Sec. Mod. S	Properties. Axis 2-2. Rad. Gyr. r
	Lb.		In.	In.	In.	In.	Sq. In.	In.³	In.	In.³	In.
10 AB	33	1001	8½	10 x ¼	4 x 3 x ¼		9.3	31.2	4.20	5.7	1.61
	40	1002		5/16	5/16		11.5	38.2	4.18	7.3	1.62
	46	1003		5/16	⅜		13.0	44.2	4.22	8.7	1.67
	47	1004	10½	5/16	5 x 3½ x 5/16		13.4	44.9	4.21	11.2	2.08
	53	1005		5/16	⅜		15.3	52.2	4.23	13.5	2.13
	60	1006		5/16	7/16		17.2	59.0	4.25	15.7	2.17
	63	1007	12½	10 x ⅜	6 x 4 x ⅜		18.2	60.8	4.19	19.2	2.56
	71	1008			7/16		20.5	68.7	4.20	22.5	2.61
	79	1009			½		22.8	76.4	4.20	25.9	2.65
	86	1010			9/16		25.0	83.9	4.20	29.1	2.68
	94	1011			⅝		27.2	91.2	4.20	32.4	2.71
	101	1012			11/16		29.3	97.7	4.18	35.7	2.74
	108	1013			¾		31.5	104	4.17	39.0	2.77
	115	1014			13/16		33.6	111	4.16	42.1	2.78
	117	1015	14	8 x ½	6 x 4 x ½	14 x ⅜	33.5	115	4.24	48.1	3.17
	123	1016				7/16	35.2	120	4.24	52.2	3.22
	129	1017				½	37.0	126	4.24	56.3	3.26
	135	1018				9/16	38.8	132	4.23	60.4	3.30
	141	1019				⅝	40.5	138	4.23	64.5	3.33
	147	1020				11/16	42.2	143	4.22	68.5	3.37
*	153	1021				¾	44.0	149	4.21	72.6	3.40
	159	1022				13/16	45.8	154	4.20	76.6	3.43

* Maximum Column for Framed Connections.

Note.—For details of "Constant Dimension Columns," see Tables 163 to 171. For discussion of "Constant Dimension Columns," see page 118, Part I.

TABLE 162.—*Continued.*
CONSTANT DIMENSION COLUMNS.
AMERICAN BRIDGE COMPANY.

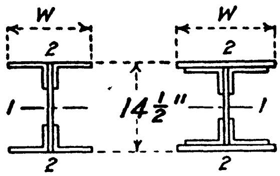


When specifying these columns, give either the Section Number and weight per foot, thus: 12 AB 121, or the Index Number thus: INDEX 1215.

Rivets ¾"

Section Number	Weight per Ft.	Index Number.	Width W	Material. Web Plate.	Material. Four Angles.	Material. Two Cover Plates.	Area.	Properties. Axis 1-1. Sec. Mod. S	Properties. Axis 1-1. Rad. Gyr. r	Properties. Axis 2-2. Sec. Mod. S	Properties. Axis 2-2. Rad. Gyr. r
	Lb.		In.	In.	In.	In.	Sq. In.	In.^{3}	In.	In.^{3}	In.
12 AB	34	1201	8½	12 x 1/4	4 x 3 x 1/4		9.8	39.5	5.01	5.7	1.56
	43	1202		5/16	5/16		12.1	48.6	5.01	7.3	1.58
	48	1203		5/16	3/8		13.7	56.0	5.06	8.7	1.63
	49	1204	10½	5/16	5 x 3½ x 5/16		14.0	56.9	5.04	11.2	2.03
	55	1205		5/16	3/8		16.0	65.9	5.07	13.5	2.08
	62	1206		5/16	7/16		17.9	74.7	5.11	15.7	2.12
	66	1207	12½	12 x 3/8	6 x 4 x 3/8		18.9	77.0	5.06	19.2	2.51
	74	1208			7/16		21.2	87.0	5.07	22.6	2.56
	81	1209			1/2		23.5	96.8	5.08	25.8	2.61
	89	1210			9/16		25.7	106	5.09	29.1	2.64
	96	1211			5/8		27.9	116	5.10	32.2	2.67
	104	1212			11/16		30.1	124	5.08	35.7	2.70
	111	1213			3/4		32.3	133	5.08	39.0	2.73
	118	1214			13/16		34.4	141	5.07	42.1	2.75
	121	1215	14	10 x 1/2	6 x 4 x 1/2	14 x 3/8	34.5	145	5.12	48.1	3.12
	127	1216				7/16	36.3	152	5.12	52.1	3.17
	132	1217				1/2	38.0	160	5.13	56.2	3.22
	138	1218				9/16	39.8	167	5.13	60.4	3.26
	144	1219				5/8	41.5	175	5.13	64.5	3.30
	152	1223	14	10 x 5/8	6 x 4 x 5/8	14 x 1/2	43.7	177	5.04	63.2	3.18
	158	1224				9/16	45.4	184	5.03	67.2	3.22
	164	1225				5/8	47.2	191	5.03	71.3	3.25
	170	1226				11/16	48.9	198	5.03	75.3	3.28
	176	1227				3/4	50.7	204	5.03	79.4	3.31
	182	1228				13/16	52.4	210	5.02	83.4	3.34
	188	1229				7/8	54.2	216	5.00	87.5	3.36
	194	1230				15/16	55.9	222	4.99	91.6	3.38
	195	1231				1	57.7	228	4.98	95.6	3.40

TABLE 162.—*Continued.*
Constant Dimension Columns.
American Bridge Company.

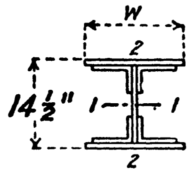


When specifying these columns, give either the Section Number and weight per foot, thus: 14 AB 125, or the Index Number thus: INDEX 1415.

Rivets ⅞″

Section Number.	Weight per Ft.	Index Number.	Width *W*	Material. Web Plate.	Material. Four Angles.	Material. Two Cover Plates.	Area.	Properties. Axis 1-1. Sec. Mod. *S*	Properties. Axis 1-1. Rad. Gyr. *r*	Properties. Axis 2-2. Sec. Mod. *S*	Properties. Axis 2-2. Rad. Gyr. *r*
	Lb.		In.	In.	In.	In.	Sq. In.	In.³	In.	In.³	In.
14 AB	37	1401	8½	14 x ¼	4 x 3 x ¼		10.3	48.2	5.83	5.7	1.52
	45	1402		5/16	5/16		12.7	59.3	5.82	7.3	1.54
	50	1403		5/16	⅜		14.3	68.3	5.88	8.7	1.59
	51	1404	10½	5/16	5 x 3½ x 5/16		14.6	69.3	5.88	11.2	1.99
	58	1405		5/16	⅜		16.6	80.3	5.92	13.5	2.04
	64	1406		5/16	7/16		18.5	90.8	5.96	15.7	2.09
	69	1407	12½	14 x ⅜	6 x 4 x ⅜		19.7	94.0	5.89	19.2	2.46
	77	1408			7/16		22.0	106	5.91	22.5	2.51
	84	1409			½		24.2	118	5.94	25.8	2.56
	92	1410			9/16		26.5	130	5.95	29.0	2.60
	99	1411			⅝		28.7	141	5.97	32.2	2.64
	107	1412			11/16		30.9	151	5.97	35.5	2.67
	114	1413			¾		33.0	162	5.98	38.9	2.70
	121	1414			13/16		35.1	173	5.97	42.1	2.72
	125	1415	14	12 x ½	6 x 4 x ½	14 x ⅜	35.5	176	6.00	48.1	3.08
	131	1416				7/16	37.3	185	6.01	52.2	3.13
	137	1417				½	39.0	195	6.03	56.3	3.18
	143	1418				9/16	40.8	204	6.03	60.3	3.22
	149	1419				⅝	42.5	213	6.03	64.4	3.26
	155	1420				11/16	44.3	221	6.03	68.5	3.29
	158	1423	14	12 x ⅝	6 x 4 x ⅝	14 x ½	44.9	217	5.92	63.1	3.13
	164	1424				9/16	46.7	225	5.92	67.1	3.17
	169	1425				⅝	48.4	234	5.92	71.2	3.21
	175	1426				11/16	50.2	242	5.92	75.3	3.24
	181	1427				¾	51.9	250	5.92	79.4	3.27
	187	1428				13/16	53.7	258	5.91	83.5	3.30
	193	1429				⅞	55.4	266	5.90	87.5	3.33
	199	1430				15/16	57.2	273	5.89	91.6	3.35
	201	1431		10 x ⅝		1	57.7	276	5.90	95.8	3.42
	207	1432				1 1/16	59.4	283	5.8[illegible]	100	3.43
	213	1433				1⅛	61.2	290	5.86	104	3.45
	219	1434				1 3/16	62.9	297	5.85	108	3.47
	225	1435				1¼	64.7	304	5.84	112	3.48
	231	1436				1 5/16	66.4	311	5.82	116	3.50
	237	1437				1⅜	68.2	317	5.81	120	3.51
	243	1438				1 7/16	69.9	324	5.79	124	3.53

CONSTANT DIMENSION COLUMNS.
AMERICAN BRIDGE COMPANY.



When specifying these columns, give either the Section Number and weight per foot, thus: 14 AB 249, or the Index Number thus: INDEX 1439.

Rivets ⅞"

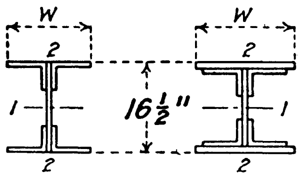
Section Number.	Weight per Ft.	Index Number.	Width W	Material. Web Plate.	Material. Four Angles.	Material. Two Cover Plates.	Area.	Properties. Axis 1-1. Sec. Mod. S	Properties. Axis 1-1. Rad. Gyr r	Properties. Axis 2-2. Sec. Mod. S	Properties. Axis 2-2. Rad. Gyr. r
	Lb.		In.	In.	In.	In.	Sq. In.	In.³	In.	In.³	In.
14 AB *	249	1439	14	10 x ⅝	6 x 4 x ⅝	14 x 1½	71.7	330	5.77	129	3.54
	255	1440				1 9/16	73.4	336	5.75	133	3.55
	260	1441				1⅝	75.2	341	5.74	137	3.57
	266	1442				1 11/16	76.9	347	5.72	141	3.58
	272	1443				1¾	78.7	352	5.70	145	3.59
	278	1444				1 13/16	80.4	357	5.68	149	3.60
	284	1445				1⅞	82.2	363	5.66	153	3.61
	290	1446				1 15/16	83.9	368	5.64	157	3.62
	296	1447				2	85.7	373	5.62	161	3.63
	165	1450	16	12 x ⅝	6 x 4 x ⅝	16 x ½	46.9	231	5.96	69.2	3.44
	171	1451				9/16	48.9	240	5.96	74.5	3.49
	178	1452				⅝	50.9	250	5.97	79.9	3.54
	185	1453				11/16	52.9	260	5.97	85.3	3.59
	192	1454				¾	54.9	270	5.97	90.6	3.63
	198	1455				13/16	56.9	279	5.96	96.0	3.67
	205	1456				⅞	58.9	288	5.96	101	3.71
	212	1457				15/16	60.9	297	5.95	107	3.74
	215	1458		10 x ⅝		1	61.7	301	5.96	112	3.82
	221	1459				1 1/16	63.7	309	5.95	117	3.84
	228	1460				1⅛	65.7	318	5.94	123	3.87
	235	1461				1 3/16	67.7	326	5.92	128	3.89
	242	1462				1¼	69.7	334	5.90	133	3.91
	250	1463				1 5/16	71.7	342	5.88	138	3.93
	256	1464				1⅜	73.7	350	5.86	144	3.95
	262	1465				1 7/16	75.7	357	5.85	149	3.97
*	269	1466				1½	77.7	365	5.84	154	3.99
	276	1467				1 9/16	79.7	371	5.82	159	4.00
	283	1468				1⅝	81.7	378	5.80	165	4.02
	289	1469				1 11/16	83.7	385	5.78	170	4.03
	296	1470				1¾	85.7	392	5.76	176	4.05
	303	1471				1 13/16	87.7	398	5.74	181	4.06
	310	1472				1⅞	89.7	405	5.73	187	4.08
	317	1473				1 15/16	91.7	411	5.71	192	4.10
	323	1474				2	93.7	418	5.69	197	4.12

* **Maximum Column for Framed Connections.**

TABLE 162.—*Continued.*

Constant Dimension Columns.

American Bridge Company.



When specifying these columns, give either the Section Number and weight per foot, thus: 16 AB 129, or the Index Number thus: INDEX 1615.

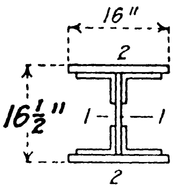
Rivets ⅞″

Section Number.	Weight per Ft.	Index Number.	Width W	Material. Web Plate.	Material. Four Angles.	Material. Two Cover Plates.	Area.	Properties. Axis 1-1. Sec. Mod. S	Properties. Axis 1-1. Rad. Gyr. r	Properties. Axis 2-2. Sec. Mod. S	Properties. Axis 2-2. Rad. Gyr. r
	Lb.		In.	In.	In.	In.	Sq. In.	In.³	In.	In.³	In.
16AB	38	1601	8½	16 x ¼	4 x 3 x ¼		10.8	57.2	6.62	5.7	1.48
	47	1602		5/16	5/16		13.4	70.6	6.60	7.3	1.51
	53	1603		5/16	⅜		14.9	81.0	6.70	8.7	1.56
	53	1604	10½	5/16	5 x 3½ x 5/16		15.2	82.4	6.69	11.2	1.95
	60	1605		5/16	⅜		17.2	95.2	6.75	13.4	2.01
	67	1606		5/16	7/16		19.1	108	6.82	15.7	2.06
	71	1607	12½	⅜	6 x 4 x ⅜		20.4	111	6.71	19.2	2.42
	79	1608			7/16		22.7	126	6.75	22.5	2.48
	87	1609			½		25.0	140	6.80	25.8	2.53
	94	1610			9/16		27.2	153	6.83	29.0	2.57
	102	1611			⅝		29.4	167	6.86	32.2	2.61
	109	1612			11/16		31.6	180	6.86	35.6	2.65
	116	1613			¾		33.8	193	6.86	39.0	2.67
	124	1614			13/16		35.9	205	6.86	42.1	2.69
	129	1615	14	14 x ½	6 x 4 x ½	14 x ⅜	36.5	208	6.87	48.1	3.04
	134	1616	16			16 x ⅜	38.0	220	6.91	52.6	3.33
	141	1617				7/16	40.0	233	6.94	67.9	3.40
	147	1618				½	42.0	246	6.96	63.2	3.47
	154	1619				9/16	44.0	259	6.96	68.6	3.53
	161	1620				⅝	46.0	271	6.97	74.0	3.59
	169	1623		14 x ⅝	6 x 4 x ⅝	16 x ½	48.2	273	6.85	69.3	3.39
	175	1624				9/16	50.2	285	6.85	74.5	3.44
	182	1625				⅝	52.2	297	6.86	79.9	3.50
	189	1626				11/16	54.2	309	6.86	85.2	3.54
	196	1627				¾	56.2	321	6.86	90.6	3.59
	203	1628				13/16	58.2	332	6.86	96.0	3.63
	210	1629				⅞	60.2	343	6.86	101	3.67
	216	1630				15/16	62.2	354	6.85	107	3.70
	219	1631		12 x ⅝		1	62.9	358	6.86	112	3.77
	226	1632				1 1/16	64.9	368	6.85	117	3.80
	232	1633				1⅛	66.9	378	6.84	123	3.83
	239	1634				1 3/16	68.9	388	6.83	128	3.86
	246	1635				1¼	70.9	398	6.82	133	3.88
	253	1636				1 5/16	72.9	408	6.80	138	3.90
	260	1637				1⅜	74.9	417	6.79	144	3.92
	266	1638				1 7/16	76.9	427	6.77	149	3.94

TABLE 162.—*Continued.*

Constant Dimension Columns.

American Bridge Company.



When specifying these columns, give either the Section Number and weight per foot, thus: 16 AB 273, or the Index Number thus: INDEX 1639.

Rivets ⅞"

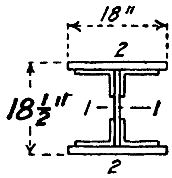
Section Number.	Weight per Ft.	Index Number.	Width	Material.			Area.	Properties.			
				Web Plate.	Four Angles.	Two Cover Plates.		Axis 1-1.		Axis 2-2.	
								Sec. Mod. S	Rad. Gyr r	Sec. Mod. S	Rad. Gyr. r
	Lb.		In.	In.	In.	In.	Sq. In.	In.³	In.	In.³	In.
16 AB	273	1639	16	12 x ⅝	6 x 4 x ⅝	16 x 1½	78.9	436	6.76	154	3.96
	280	1640				1 9/16	80.9	445	6.74	159	3.98
	287	1641				1⅝	82.9	454	6.73	165	3.99
	294	1642				1 11/16	84.9	463	6.71	170	4.01
	300	1643				1¾	86.9	471	6.69	176	4.02
	307	1644				1 13/16	88.9	479	6.67	181	4.04
	314	1645	*			1⅞	90.9	487	6.66	186	4.05
	321	1646				1 15/16	92.9	495	6.64	192	4.07
	323	1647		10 x ⅝		2	93.7	498	6.63	197	4.10
	330	1648				2 1/16	95.7	506	6.61	203	4.11
	337	1649				2⅛	97.7	513	6.59	208	4.12
	344	1650				2 3/16	99.7	520	6.57	213	4.14
	351	1651				2¼	101.7	527	6.55	219	4.15
	357	1652				2 5/16	103.7	534	6.52	224	4.16
	364	1653				2⅜	105.7	540	6.50	229	4.17
	371	1654				2 7/16	107.7	547	6.48	234	4.18
	378	1655				2½	109.7	554	6.46	240	4.19

*·Maximum Column for Framed Connections.

TABLE 162.—*Continued.*

CONSTANT DIMENSION COLUMNS.

AMERICAN BRIDGE COMPANY.



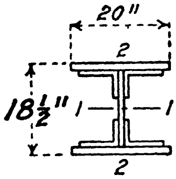
When specifying these columns, give either the Section Number and weight per foot, thus: 18 AB 276, or the Index Number thus: INDEX 1827.

Rivets 1"

Section Number.	Weight per Ft.	Index Number.	Width	Material. Web Plate.	Material. Four Angles.	Material. Two Cover Plates.	Area.	Properties. Axis 1-1. Sec. Mod. S	Properties. Axis 1-1. Rad. Gyr. r	Properties. Axis 2-2. Sec. Mod. S	Properties. Axis 2-2. Rad. Gyr. r
	Lb.		In.	In.	In.	In.	Sq. In.	In.³	In.	In.³	In.
18 AB	276	1827	18	16 x ¾	8 x 6 x ¾	18 x ¾	78.8	478	7.49	147	4.11
	283	1828				13/16	81.0	492	7.49	154	4.14
	291	1829				⅞	83.3	506	7.50	161	4.18
	299	1830				15/16	85.5	519	7.50	167	4.22
	304	1831		15 x ¾		1	87.0	527	7.48	174	4.25
	312	1832				1 1/16	89.3	540	7.47	181	4.27
	319	1833				1⅛	91.5	553	7.47	188	4.30
	327	1834				1 3/16	93.8	565	7.46	195	4.32
	335	1835				1¼	96.0	577	7.46	201	4.34
	342	1836				1 5/16	98.3	589	7.45	208	4.36
	350	1837				1⅜	100.5	601	7.44	215	4.38
	358	1838				1 7/16	102.8	613	7.43	221	4.40
	363	1839		14 x ¾		1½	104.3	620	7.43	228	4.45
	370	1840				1 9/16	106.5	631	7.40	235	4.46
	378	1841				1⅝	108.8	642	7.38	242	4.48
	386	1842				1 11/16	111.0	652	7.37	248	4.49
	393	1843				1¾	113.3	663	7.36	255	4.50
	401	1844				1 13/16	115.5	673	7.34	262	4.51
	409	1845				1⅞	117.8	684	7.33	269	4.53
	416	1846				1 15/16	120.0	694	7.31	275	4.54
	421	1847		13 x ¾		2	121.5	700	7.30	282	4.57
	429	1848				2 1/16	123.8	710	7.29	289	4.58
*	437	1849				2⅛	126.0	719	7.27	296	4.60
	444	1850				2 3/16	128.3	728	7.25	302	4.61
	452	1851				2¼	130.5	738	7.23	309	4.62
	460	1852				2 5/16	132.8	746	7.21	316	4.63
	467	1853				2⅜	135.0	755	7.19	323	4.64
	475	1854				2 7/16	137.3	763	7.17	329	4.65
	480	1855		12 x ¾		2½	138.8	769	7.16	336	4.68
	488	1856				2 9/16	141.0	777	7.14	343	4.69
	495	1857				2⅝	143.3	785	7.13	350	4.70
	503	1858				2 11/16	145.5	793	7.11	356	4.71
	511	1859				2¾	147.8	801	7.09	363	4.71
	518	1860				2 13/16	150.0	808	7.07	370	4.72
	526	1861				2⅞	152.3	815	7.05	377	4.73
	534	1862				2 15/16	154.5	823	7.03	383	4.73
	541	1863				3	156.8	830	7.01	390	4.74

[1] **Maximum Column for Framed Connections.**

TABLE 162.—*Continued.*
CONSTANT DIMENSION COLUMNS.
AMERICAN BRIDGE COMPANY.



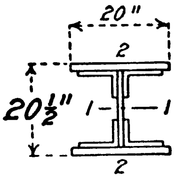
When specifying these columns, give either the Section Number and weight per foot, thus: 18 AB 286, or the Index Number thus: INDEX 1867.

Rivets 1"

Section Number.	Weight per Ft.	Index Number.	Width	Material. Web Plate.	Material. Four Angles.	Material. Two Cover Plates.	Area.	Properties. Axis 1–1. Sec. Mod. *S*	Properties. Axis 1–1. Rad. Gyr. *r*	Properties. Axis 2–2. Sec. Mod. *S*	Properties. Axis 2–2. Rad. Gyr. *r*
	Lb.		In.	In.	In.	In.	Sq. In.	In.^{3}	In.	In.^{3}	In.
18 AB	286	1867	20	16 x ¾	8 x 6 x ¾	20 x ¾	81.8	501	7.55	160	4.42
	295	1868				13/16	84.3	520	7.55	168	4.47
	303	1869				⅞	86.8	535	7.55	177	4.52
	312	1870				15/16	89.3	550	7.54	185	4.56
	318	1871		15 x ¾		1	91.0	560	7.55	193	4.60
	326	1872				1 1/16	93.5	575	7.54	201	4.64
	336	1873				1 ⅛	96.0	589	7.54	210	4.67
	343	1874				1 3/16	98.5	604	7.53	218	4.70
	352	1875				1 ¼	101.0	618	7.52	226	4.73
	360	1876				1 5/16	103.5	631	7.51	235	4.76
	369	1877				1 ⅜	106.0	645	7.50	243	4.79
	377	1878				1 7/16	108.5	658	7.49	251	4.82
	383	1879		14 x ¾		1 ½	110.3	667	7.49	260	4.86
	392	1880				1 9/16	112.8	679	7.47	268	4.88
	400	1881				1 ⅝	115.3	691	7.46	276	4.91
	410	1882				1 11/16	117.8	703	7.44	284	4.92
	417	1883				1 ¾	120.3	716	7.43	293	4.94
	426	1884				1 13/16	122.8	728	7.41	301	4.96
	434	1885				1 ⅞	125.3	740	7.40	310	4.98
	443	1886				1 15/16	127.8	752	7.38	318	4.99
	449	1887		13 x ¾		2	129.5	759	7.36	326	5.02
	457	1888				2 1/16	132.0	770	7.35	335	5.04
	*466	1889				2 ⅛	134.5	781	7.33	343	5.05
	474	1890				2 3/16	137.0	791	7.31	351	5.06
	483	1891				2 ¼	139.5	802	7.29	360	5.08
	491	1892				2 5/16	142.0	812	7.27	368	5.09
	500	1893				2 ⅜	144.5	822	7.25	376	5.10
	508	1894				2 7/16	147.0	832	7.23	385	5.11
	514	1895		12 x ¾		2 ½	148.8	838	7.22	393	5.14
	522	1896				2 9/16	151.3	847	7.20	401	5.15
	531	1897				2 ⅝	153.8	857	7.18	410	5.16
	540	1898				2 11/16	156.3	866	7.16	418	5.17
	548	1899				2 ¾	158.8	875	7.14	426	5.18
	557	18100				2 13/16	161.3	884	7.12	434	5.19
	565	18101				2 ⅞	163.8	893	7.10	443	5.20
	574	18102				2 15/16	166.3	901	7.08	451	5.21
	582	18103				3	168.8	910	7.06	460	5.22

* **Maximum Column for Framed Connections.**

CONSTANT DIMENSION COLUMNS.
AMERICAN BRIDGE COMPANY.



When specifying these columns, give either the Section Number and weight per foot, thus: 20 AB 357, or the Index Number thus: INDEX 2035.

Rivets 1"

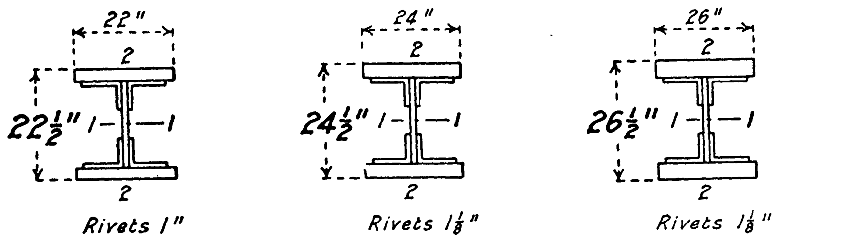
Section Number.	Weight per Ft.	Index Number.	Width	Material. Web Plate.	Material. Four Angles.	Material. Two Cover Plates.	Area.	Properties. Axis 1-1. Sec. Mod. S	Properties. Axis 1-1. Rad. Gyr. r	Properties. Axis 2-2. Sec. Mod. S	Properties. Axis 2-2. Rad. Gyr. r
	Lb.		In.	In.	In.	In.	Sq. In.	In.³	In.	In.³	In.
20 AB	357	2035	20	17 x ¾	8 x 6 x ¾	20 x 1¼	102.5	709	8.42	226	4.70
	365	2036				1 5/16	105.0	725	8.41	235	4.73
	374	2037				1⅜	107.5	741	8.40	243	4.75
	382	2038				1 7/16	110.0	757	8.39	251	4.78
	388	2039		16 x ¾		1½	111.8	767	8.38	260	4.81
	397	2040				1 9/16	114.3	783	8.37	268	4.83
	405	2041				1⅝	116.8	799	8.36	276	4.86
	414	2042				1 11/16	119.3	812	8.34	284	4.88
	422	2043				1¾	121.8	825	8.33	293	4.90
	431	2044				1 13/16	124.3	839	8.31	301	4.92
	439	2045				1⅞	126.8	854	8.30	310	4.94
	448	2046				1 15/16	129.3	867	8.29	318	4.96
	454	2047		15 x ¾		2	131.0	876	8.28	326	4.99
	462	2048				2 1/16	133.5	890	8.27	335	5.00
	471	2049				2⅛	136.0	903	8.25	343	5.02
	479	2050				2 3/16	138.5	916	8.24	351	5.03
	488	2051				2¼	141.0	929	8.22	360	5.05
	496	2052				2 5/16	143.5	941	8.20	368	5.06
	505	2053				2⅜	146.0	952	8.18	376	5.08
	513	2054				2 7/16	148.5	964	8.16	385	5.09
	519	2055		14 x ¾		2½	150.3	973	8.16	393	5.12
	528	2056				2 9/16	152.8	984	8.14	401	5.13
	* 536	2057				2⅝	155.3	996	8.12	410	5.14
	545	2058				2 11/16	157.8	1 007	8.10	418	5.15
	553	2059				2¾	160.3	1 018	8.08	426	5.16
	562	2060				2 13/16	162.8	1 028	8.06	434	5.17
	570	2061				2⅞	165.3	1 038	8.04	443	5.18
	579	2062				2 15/16	167.8	1 048	8.02	451	5.19
	585	2063		13 x ¾		3	169.5	1 056	7.99	460	5.21
	593	2064				3 1/16	172.0	1 066	7.97	468	5.22
	602	2065				3⅛	174.5	1 076	7.95	476	5.22
	610	2066				3 3/16	177.0	1 085	7.92	485	5.23
	619	2067				3¼	179.5	1 094	7.90	493	5.24
	627	2068				3 5/16	182.0	1 104	7.88	501	5.25
	636	2069				3⅜	184.5	1 115	7.87	510	5.25
	644	2070				3 7/16	187.0	1 124	7.85	518	5.26
	652	2071				3½	189.5	1 133	7.83	527	5.27

* Maximum Column for Framed Connections.

TABLE 162.—*Continued.*

CONSTANT DIMENSION COLUMNS.

AMERICAN BRIDGE COMPANY.



Section Number.	Weight per Ft.	Index Number.	Width	Material. Web Plate.	Material. Four Angles.	Material. Two Cover Plates.	Area.	Properties. Axis 1-1. Sec. Mod. S	Properties. Axis 1-1. Rad. Gyr. r	Properties. Axis 2-2. Sec. Mod. S	Properties. Axis 2-2. Rad. Gyr. r
	Lb.		In.	In.	In.	In.	Sq. In	In.³	In.	In.³	In.
22 AB	508	2213	22	18 x 1	8 x 6 x 1	22 x 1¾	147	1 080	9.09	359	5.18
	545	2217		"		2	158	1 148	9.04	399	5.27
	579	2221		17 x 1		2¼	168	1 206	8.98	439	5.36
	617	2225		"		2½	179	1 266	8.92	480	5.43
	651	2229	*	16 x 1		2¾	189	1 316	8.85	520	5.50
	688	2233		"		3	200	1 369	8.78	560	5.55
	721	2237		15 x 1		3¼	210	1 413	8.70	601	5.61
	759	2241		"		3½	221	1 460	8.62	641	5.65
	793	2245		14 x 1		3¾	231	1 498	8.54	681	5.70
	830	2249		"		4	242	1 538	8.46	722	5.73
24 AB	685	2421	24	19 x 2	8 x 6 x 1	24 x 2¼	198	1 496	9.62	518	5.60
	725	2425		"		2½	210	1 575	9.58	566	5.69
	760	2429		18 x 2		2¾	220	1 635	9.54	614	5.79
	800	2433		"		3	232	1 705	9.49	662	5.85
	834	2437	*	17 x 2		3¼	242	1 757	9.43	710	5.93
	875	2441		"		3½	254	1 819	9.37	758	5.98
	910	2445		16 x 2		3¾	264	1 866	9.31	806	6.05
	950	2449		"		4	276	1 921	9.23	854	6.09
	984	2453		15 x 2		4¼	286	1 961	9.17	902	6.15
	1 025	2457		"		4½	298	2 009	9.09	950	6.18
	1 059	2461		14 x 2		4¾	308	2 045	9.02	998	6.23
	1 100	2465		"		5	320	2 087	8.94	1 046	6.26
26 AB *	923	2645	24	18 x 2	8 x 6 x 1	24 x 3¾	268	2 101	10.19	806	6.01
	963	2649		"		4	280	2 165	10.12	854	6.05
	997	2653		17 x 2		4¼	290	2 214	10.06	902	6.11
	1 038	2657		"		4½	302	2 271	9.99	950	6.14
	1 072	2661		16 x 2		4¾	312	2 314	9.92	998	6.19
	1 113	2665		"		5	324	2 365	9.84	1 046	6.22
	1 147	2669		15 x 2		5¼	334	2 403	9.76	1 094	6.27
	1 188	2673		"		5½	346	2 447	9.68	1 142	6.29
	1 222	2677		14 x 2		5¾	356	2 480	9.60	1 190	6.33
	1 263	2681		"		6	368	2 518	9.52	1 238	6.35

* Maximum Column for Framed Connections.

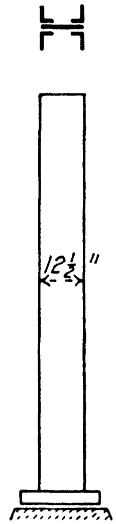
TABLE 163.

Constant Dimension Columns.

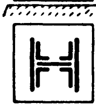
American Bridge Company.

15 Story Hotel

Story	Story Height	Load in 1000 lb.	Column
Roof			
Loft	8'	45	12AB43
12	10'	91	
11	"	135	
10	"	179	12AB62
9	"	223	
8	"	266	12AB81
7	"	308	
6	"	349	12AB104
5	"	390	
4	"	430	12AB121
3	"	470	
2	16'	509	12AB152
1	13'	548	
Bmt.	16'	622	12AB182
Sub.B	12'	689	

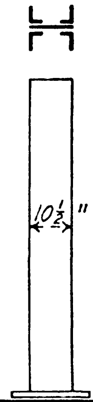


Rolled Steel Slab



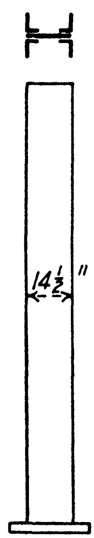
12 Story Apartment House

Story	Story Height	Load in 1000 lb.	Column
Roof			
Loft	8'	36	10 AB 33
10	11'	70	
9	"	104	10 AB 46
8	"	138	
7	"	171	10 AB 60
6	"	204	
5	"	236	10 AB 79
4	"	267	
3	"	298	10 AB 94
2	"	328	
1	17'	358	10 AB 115
Bmt.	12'	415	



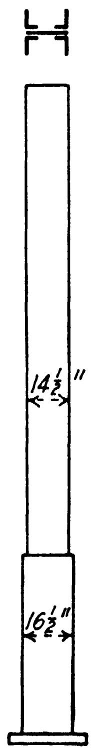
17 Story Office Building

Story	Story Height	Load in 1000 lb.	Column
Roof			
16	12'	53	14AB37
15	13'	104	14 AB 51
14	14'	155	
13	13'	204	14 AB 77
12	"	253	
11	"	300	14 AB 99
10	"	347	
9	"	398	14AB125
8	"	437	
7	"	480	14AB143
6	"	522	
5	"	564	14 AB169
4	"	606	
3	"	645	14AB193
2	15'	690	
1	17'	736	14AB213
Bmt.	12'	792	



25 Story Office Building

Story	Story Height	Load in 1000 lb.	Column
Roof			
23	12'	53	14 AB 45
22	"	104	
21	"	155	14 AB 64
20	"	204	
19	"	253	14 AB 84
18	"	300	
17	"	347	14AB107
16	"	392	
15	"	437	14AB131
14	"	480	
13	"	522	14AB155
12	"	564	
11	"	606	14AB171
10	"	648	
9	"	690	14AB192
8	"	732	
7	"	774	14AB212
6	"	816	
5	"	858	16AB232
4	"	900	
3	"	942	16AB253
2	"	984	
1	18'	1026	16AB294
Bmt.	15'	1082	
Sub B	12'	1132	



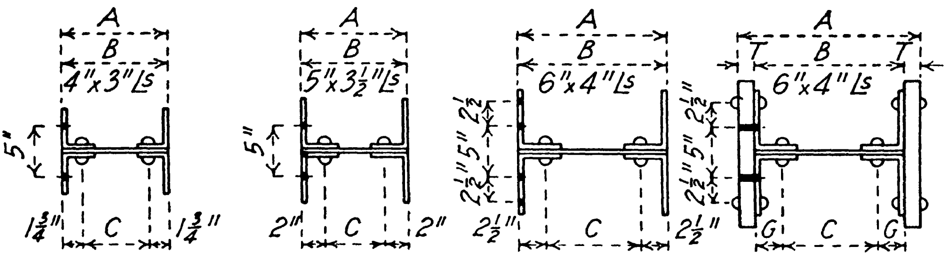
Note.—For discussion of "Constant Dimension Columns," see page 118, Part I.

TABLE 164.

Gages for Constant Dimension Columns.

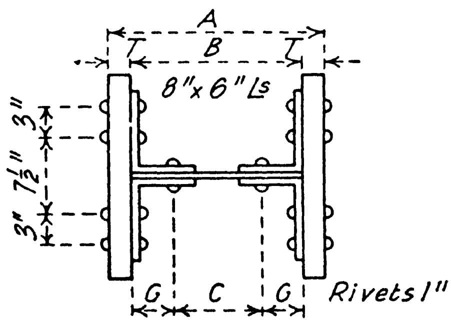
Typical Gages; Interpolate for Intermediate Thicknesses.

American Bridge Company.



Rivets $\frac{7}{8}$" Dimensions in Inches

Angles	Cover T, in.	Gage G, in.	Distance out to out = A: 10½" B	10½" C	12½" B	12½" C	14½" B	14½" C	16½" B	16½" C
4 x 3	None	1¾	10½	7	12½	9	14½	11	16½	13
5 x 3½	None	2	10½	6½	12½	8½	14½	10½	16½	12½
6 x 4	None	2½	10½	5½	12½	7½	14½	9½	16½	11½
6 x 4	$\frac{3}{8}$	2$\frac{3}{8}$	9¾	5	11¾	7	13¾	9	15¾	11
	$\frac{7}{16}$	2$\frac{5}{16}$	9$\frac{5}{8}$	5	11$\frac{5}{8}$	7	13$\frac{5}{8}$	9	15$\frac{5}{8}$	11
	½	2¾	9½	4	11½	6	13½	8	15½	10
	$\frac{15}{16}$	2$\frac{5}{16}$	8$\frac{5}{8}$	4	10$\frac{5}{8}$	6	12$\frac{5}{8}$	8	14$\frac{5}{8}$	10
	1	2¾					12½	7	14½	9
	1$\frac{7}{16}$	2$\frac{5}{16}$					11$\frac{5}{8}$	7	13$\frac{5}{8}$	9
	1½	2¾					11½	6	13½	8
	1$\frac{15}{16}$	2$\frac{5}{16}$					10$\frac{5}{8}$	6	12$\frac{5}{8}$	8
	2	2¾							12½	7
	2$\frac{7}{16}$	2$\frac{5}{16}$							11$\frac{5}{8}$	7



In establishing B and C when T is 2½" or over, $\frac{1}{16}$" has been added to allow for packing.

Cover T, in.	Gage G, in.	Distance out to out = A: 18½" B	18½" C	20½" B	20½" C
½	3¼	17½	11	19½	13
$\frac{15}{16}$	2$\frac{13}{16}$	16$\frac{5}{8}$	11	18$\frac{5}{8}$	13
1	3¼	16½	10	18½	12
1$\frac{7}{16}$	2$\frac{13}{16}$	15$\frac{5}{8}$	10	17$\frac{5}{8}$	12
1½	3¼	15½	9	17½	11
1$\frac{15}{16}$	2$\frac{13}{16}$	14$\frac{5}{8}$	9	16$\frac{5}{8}$	11
2	3¼	14½	8	16½	10
2$\frac{7}{16}$	2$\frac{13}{16}$	13$\frac{5}{8}$	8	15$\frac{5}{8}$	10
2½	3$\frac{3}{16}$	13$\frac{3}{8}$	7	15$\frac{3}{8}$	9
2$\frac{15}{16}$	2¾	12½	7	14½	9
3	3$\frac{3}{16}$			14$\frac{3}{8}$	8
3$\frac{7}{16}$	2¾			13½	8

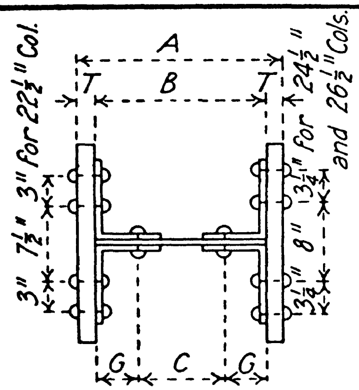
See note Table 163.

TABLE 165.

Gages and Limits for Constant Dimension Columns.

Typical Gages; Interpolate for Intermediate Thicknesses.

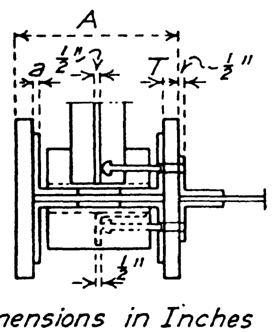
American Bridge Company.



Rivets 1" for 22½" Cols.
Rivets 1⅛" for 24½", 26½" Col.
Dimensions in Inches.
In establishing B and C, where T is 2½" or over, $\frac{1}{16}$" has been added to T to allow for packing

Cover T, in.	Gage G, in.	Distance out to out = A: 22½" B	22½" C	24½" B	24½" C	26½" B	26½" C
1½	3¾	19½	12	21½	14	23½	16
1 15/16	3 5/16	18⅝	12	20⅝	14	22⅝	16
2	3¾	18½	11	20½	13	22½	15
2 7/16	3 5/16	17⅝	11	19⅝	13	21⅝	15
2½	3 11/16	17⅜	10	19⅜	12	21⅜	14
2 15/16	3¼	16½	10	18½	12	20½	14
3	3 11/16	16⅜	9	18⅜	11	20⅜	13
3 7/16	3¼	15½	9	17½	11	19½	13
3½	3 11/16	15⅜	8	17⅜	10	19⅜	12
3 15/16	3¼	14½	8	16½	10	18½	12
4	3 11/16			16⅜	9	18⅜	11
4 7/16	3¼			15½	9	17½	11
4½	3 11/16			15⅜	8	17⅜	10
4 15/16	3¼			14½	8	16½	10
5	3 11/16					16⅜	9
5 7/16	3¼					15½	9
5½	3 11/16					15⅜	8
6	3 3/16					14⅜	8

LIMITS FOR INSERTING FIELD RIVETS



Dimensions in Inches

A	a	Diameter of Rivet: ¾"	⅞"	1"	1⅛"	1¼"
10½	½	¾	⅝			
12½	⅝	1⅛	1			
14½	⅝	1⅝	1½			
16½	⅝	2	1⅞			
18½	¾		2¼	2⅛		
20½	¾		2¾	2⅝		
22½	1		2⅞	2¾	2¾	
24½	1			3¼	3⅛	3
26½	1			3¾	3⅝	3½

Table gives the maximum thickness of cover plates T that will allow field rivets to be inserted when there is a member ½" thick on center line of web face, and ½" framing angles on cover face.
If covers exceed tabulated thickness, use seated connections, shop rivet framing angles on cover face, or select wider column.
Where conditions vary from the assumptions made here, riveting must receive special study.

See note Table 163.

TABLE 166.

Constant Dimension Columns.

Standard Column Splices.

American Bridge Company.

	Upper Section	$10\frac{1}{2}$ in. $12\frac{1}{2}$ in. $14\frac{1}{2}$ in. $16\frac{1}{2}$ in.	$10\frac{1}{2}$ in. $12\frac{1}{2}$ in. Light
	Column Angles Column Rivets	$4'' \times 3''$ $5'' \times 3\frac{1}{2}''$ $\frac{3}{4}''$ or $\frac{7}{8}''$	$6'' \times 4''$ $\frac{7}{8}''$
	Splice Plates A	$\frac{3}{8}'' \times 1'-6\frac{1}{2}''$	$\frac{3}{8}'' \times 1'-6\frac{1}{2}''$
	Bearing " B	--------	$\frac{7}{8}'' \times 1'-3\frac{1}{4}''$
Web splice angles are not used on $10\frac{1}{2}''$ columns.	" " C	----------	$1'' \times 2'-3\frac{1}{4}''$
	Splice Angles	$3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$	$3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$

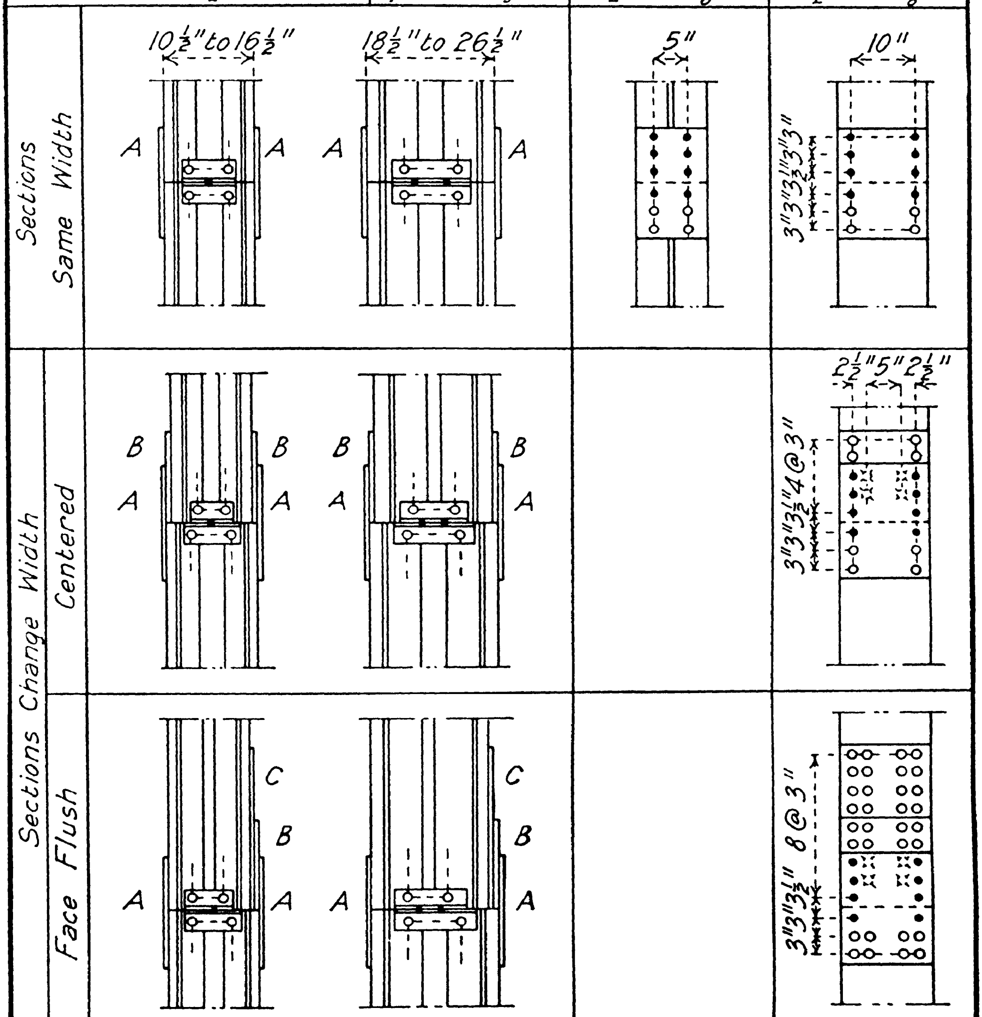


TABLE 167.

Constant Dimension Columns.

Standard Column Splices.

American Bridge Company.

$12\frac{1}{2}$ in. Heavy $14\frac{1}{2}$ in. $16\frac{1}{2}$ in. Light	$16\frac{1}{2}$ in. Heavy	$18\frac{1}{2}$ in. $20\frac{1}{2}$ in. $22\frac{1}{2}$ in.	$24\frac{1}{2}$ in. $26\frac{1}{2}$ in.
$6'' \times 4''$ $\frac{7}{8}''$	$6'' \times 4''$ $\frac{7}{8}''$	$8'' \times 6''$ $1''$	$8'' \times 6''$ $1\frac{1}{8}''$
$\frac{7}{16}'' \times 2'\text{-}0\frac{1}{2}''$ $\frac{7}{8}'' \times 1'\text{-}3\frac{3}{4}''$ $1'' \times 2'\text{-}3\frac{3}{4}''$ $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$	$\frac{1}{2}'' \times 2'\text{-}0\frac{1}{2}''$ $\frac{7}{8}'' \times 1'\text{-}6\frac{1}{4}''$ $1'' \times 2'\text{-}6\frac{1}{4}''$ $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$	$\frac{5}{8}'' \times 2'\text{-}0\frac{1}{2}''$ $\frac{7}{8}'' \times 1'\text{-}6\frac{1}{4}''$ $1'' \times 2'\text{-}6\frac{1}{4}''$ $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	$\frac{3}{4}'' \times 2'\text{-}4\frac{1}{2}''$ $\frac{7}{8}'' \times 1'\text{-}9\frac{1}{4}''$ $1'' \times 2'\text{-}11\frac{1}{4}''$ $4'' \times 4'' \times \frac{1}{2}''$

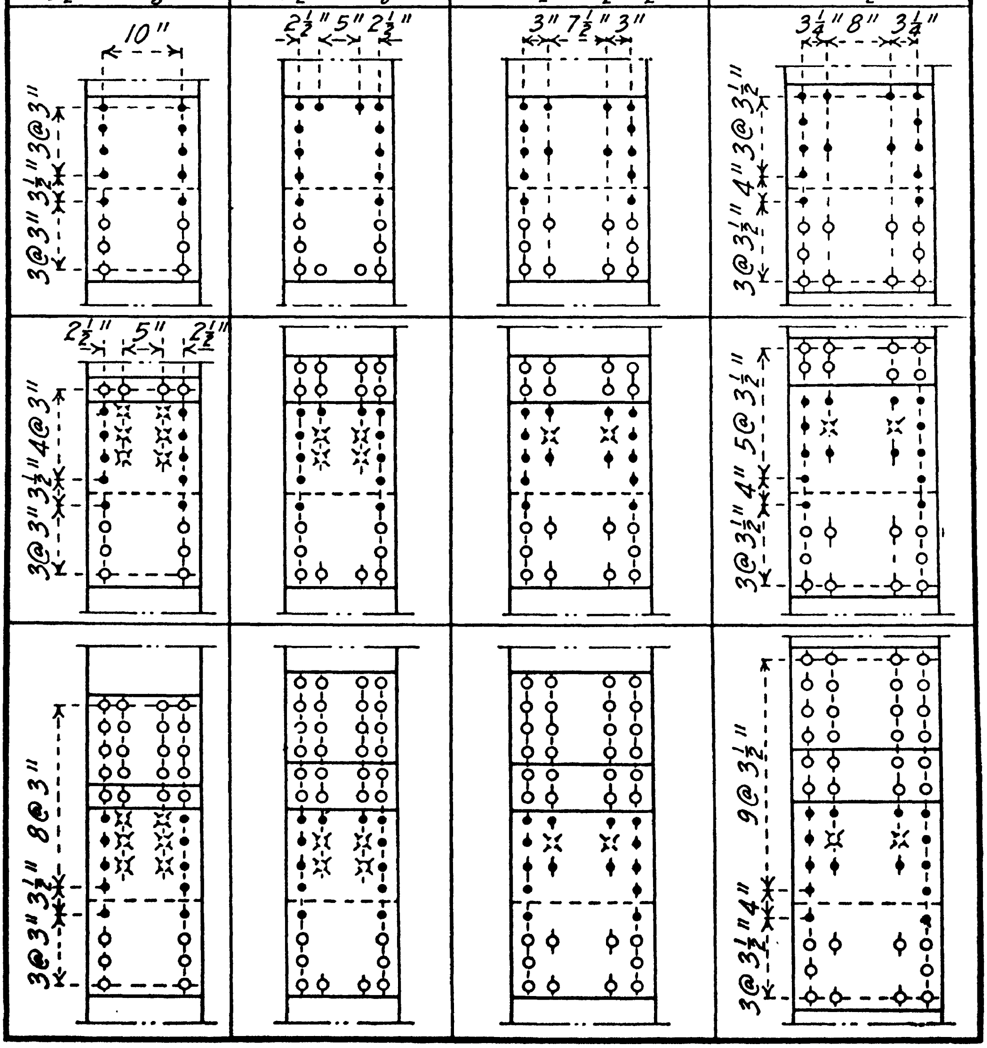


TABLE 168.

Typical Details for Constant Dimension Columns.

Connections to Interior Columns.

American Bridge Company.

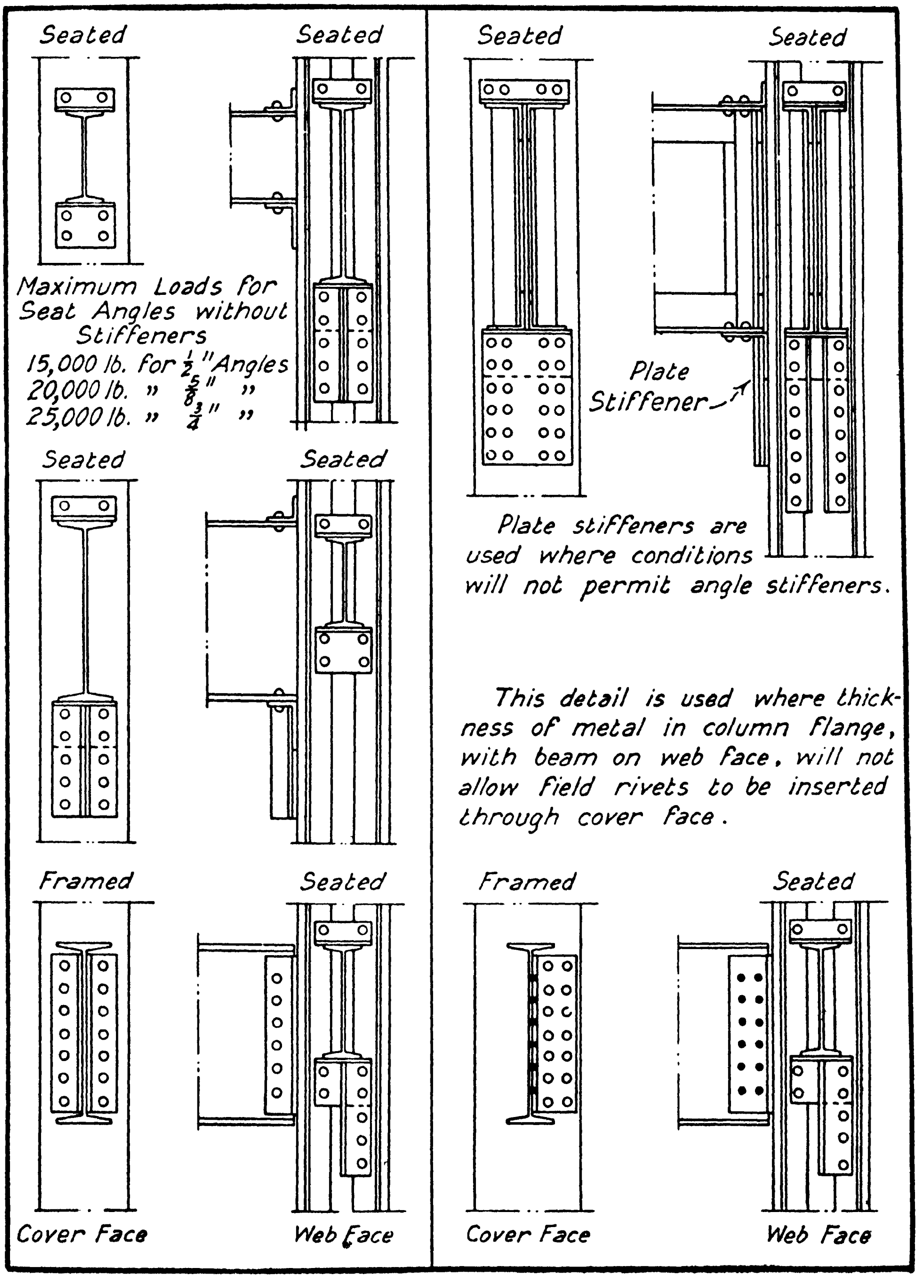


TABLE 169.

TYPICAL DETAILS FOR CONSTANT DIMENSION COLUMNS.

CONNECTIONS TO INTERIOR COLUMNS—WIND BRACING.

American Bridge Company.

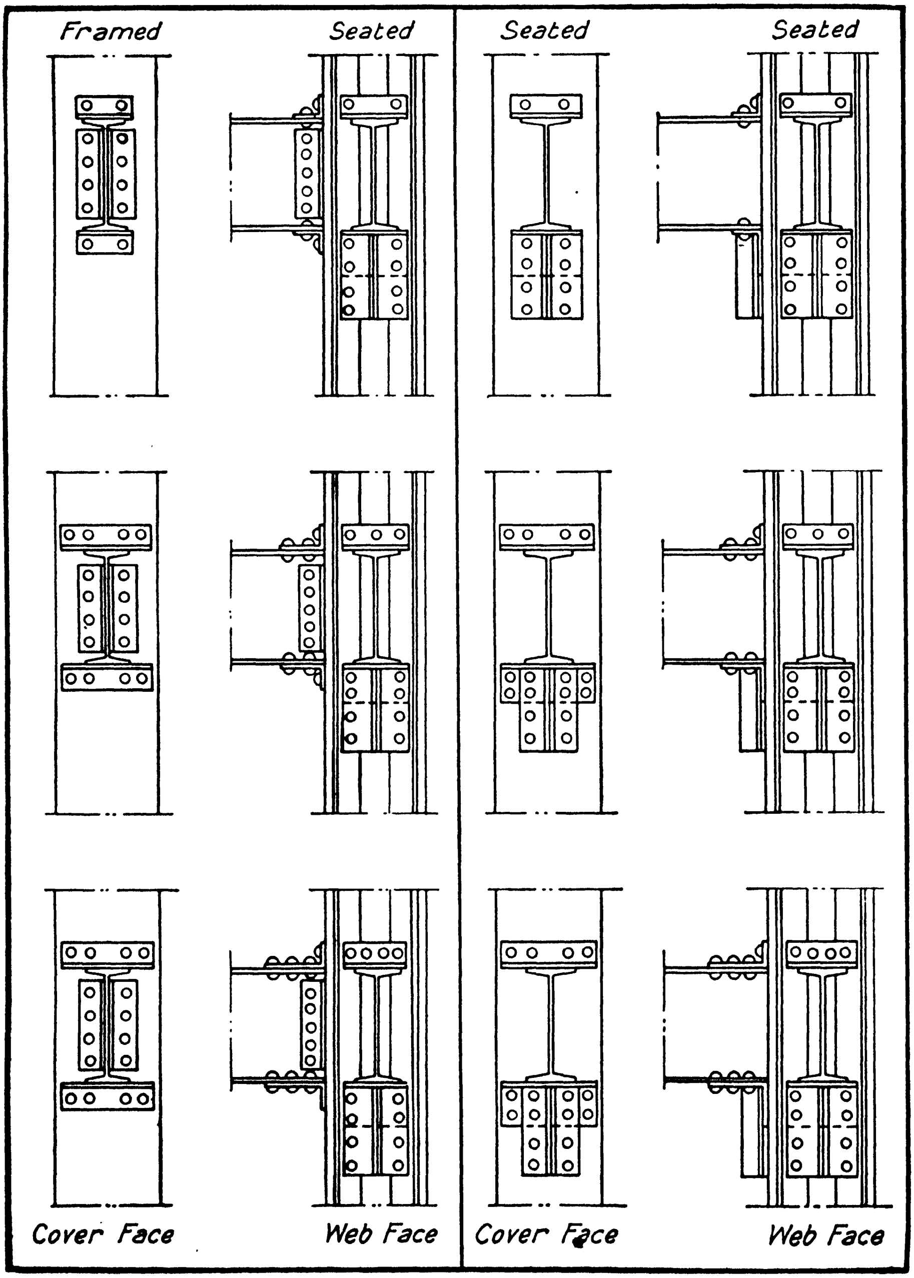


TABLE 170.

Typical Details for Constant Dimension Columns.
Location of Wall Columns and Spandrel Beams.
American Bridge Company.

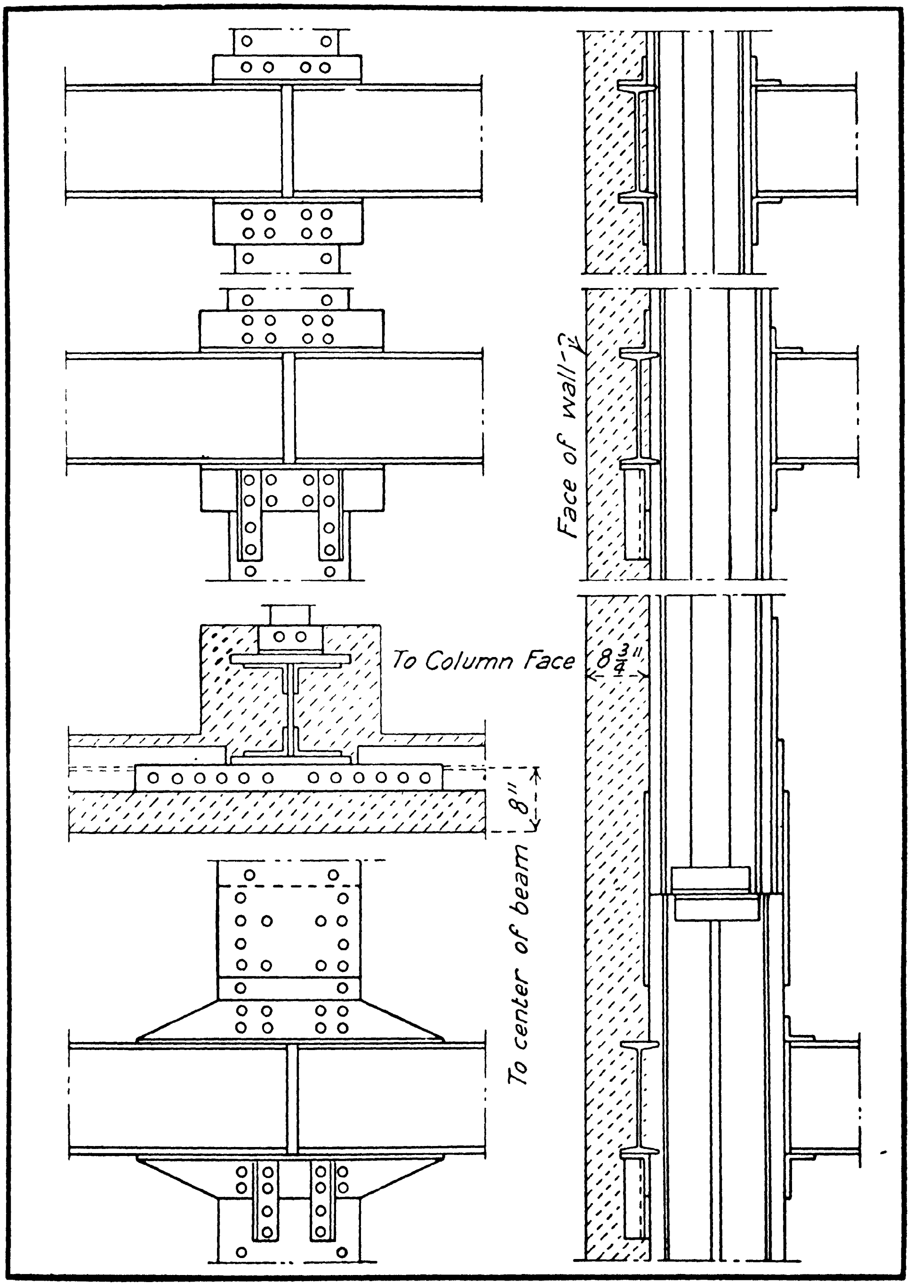


TABLE 171.

Typical Details for Constant Dimension Columns.
Location of Wall Columns, Wall Beams, and Wind Bracing.

American Bridge Company.

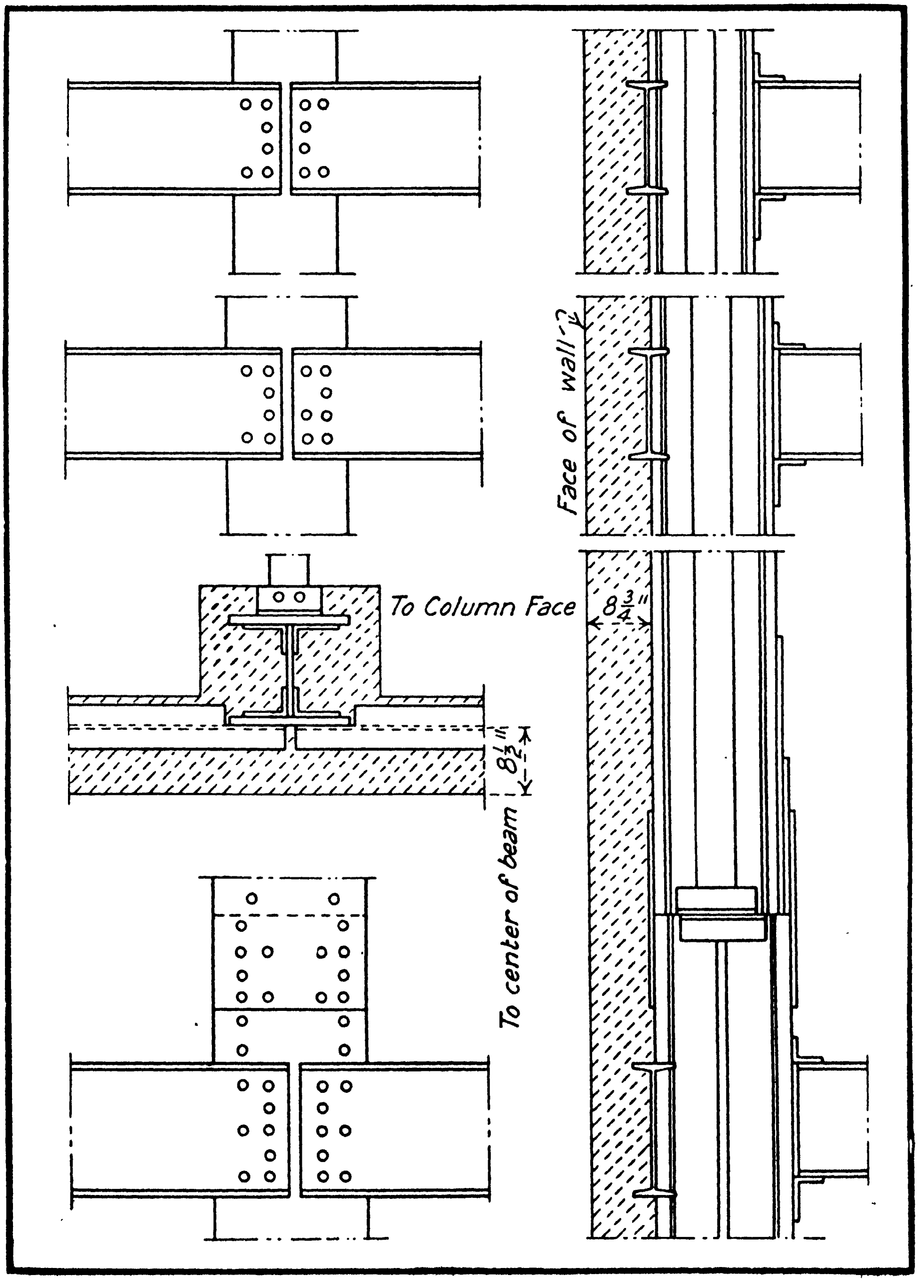


TABLE 172

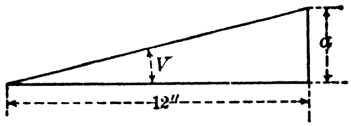
Decimal Parts of a Foot and Inch

Ins.	Decimal Parts of a Foot 0″	1″	2″	3″	4″	5″	6″	7″	8″	9″	10″	11″	Decimal Parts of an Inch	
	.0	.0833	.1667	.2500	.3333	.4167	.5000	.5833	.6667	.7500	.8333	.9167		
1/32	.0026	.0859	.1693	.2526	.3359	.4193	.5026	.5859	.6693	.7526	.8359	.9193	1/32	.0313
1/16	.0052	.0885	.1719	.2552	.3385	.4219	.5052	.5885	.6719	.7552	.8385	.9219	1/16	.0625
3/32	.0078	.0911	.1745	.2578	.3411	.4245	.5078	.5911	.6745	.7578	.8411	.9245	3/32	.0938
1/8	.0104	.0938	.1771	.2604	.3438	.4271	.5104	.5938	.6771	.7604	.8438	.9271	1/8	.125
5/32	.0130	.0964	.1797	.2630	.3464	.4297	.5130	.5964	.6797	.7630	.8464	.9297	5/32	.1563
3/16	.0156	.0990	.1823	.2656	.3490	.4323	.5156	.5990	.6823	.7656	.8490	.9323	3/16	.1875
7/32	.0182	.1016	.1849	.2682	.3516	.4349	.5182	.6016	.6849	.7682	.8516	.9349	7/32	.2188
1/4	.0208	.1042	.1875	.2708	.3542	.4375	.5208	.6042	.6875	.7708	.8542	.9375	1/4	.25
9/32	.0234	.1068	.1901	.2734	.3568	.4401	.5234	.6068	.6901	.7734	.8568	.9401	9/32	.2813
5/16	.0260	.1094	.1927	.2760	.3594	.4427	.5260	.6094	.6927	.7760	.8594	.9427	5/16	.3125
11/32	.0286	.1120	.1953	.2786	.3620	.4453	.5286	.6120	.6953	.7786	.8620	.9453	11/32	.3438
3/8	.0313	.1146	.1979	.2813	.3646	.4479	.5313	.6146	.6979	.7813	.8646	.9479	3/8	.375
13/32	.0339	.1172	.2005	.2839	.3672	.4505	.5339	.6172	.7005	.7839	.8672	.9505	13/32	.4063
7/16	.0365	.1198	.2031	.2865	.3698	.4531	.5365	.6198	.7031	.7865	.8698	.9531	7/16	.4375
15/32	.0391	.1224	.2057	.2891	.3724	.4557	.5391	.6224	.7057	.7891	.8724	.9557	15/32	.4688
1/2	.0417	.1250	.2083	.2917	.3750	.4583	.5417	.6250	.7083	.7917	.8750	.9583	1/2	.5
17/32	.0443	.1276	.2109	.2943	.3776	.4609	.5443	.6276	.7109	.7943	.8776	.9609	17/32	.5313
9/16	.0469	.1302	.2135	.2969	.3802	.4635	.5469	.6302	.7135	.7969	.8802	.9635	9/16	.5625
19/32	.0495	.1328	.2161	.2995	.3828	.4661	.5495	.6328	.7161	.7995	.8828	.9661	19/32	.5938
5/8	.0521	.1354	.2188	.3021	.3854	.4688	.5521	.6354	.7188	.8021	.8854	.9688	5/8	.625
21/32	.0547	.1380	.2214	.3047	.3880	.4714	.5547	.6380	.7214	.8047	.8880	.9714	21/32	.6563
11/16	.0573	.1406	.2240	.3073	.3906	.4740	.5573	.6406	.7240	.8073	.8906	.9740	11/16	.6875
23/32	.0599	.1432	.2266	.3099	.3932	.4766	.5599	.6432	.7266	.8099	.8932	.9766	23/32	.7188
3/4	.0625	.1458	.2292	.3125	.3958	.4792	.5625	.6458	.7292	.8125	.8958	.9792	3/4	.75
25/32	.0651	.1484	.2318	.3151	.3984	.4818	.5651	.6484	.7318	.8151	.8984	.9818	25/32	.7813
13/16	.0677	.1510	.2344	.3177	.4010	.4844	.5677	.6510	.7344	.8177	.9010	.9844	13/16	.8125
27/32	.0703	.1536	.2370	.3203	.4036	.4870	.5703	.6536	.7370	.8203	.9036	.9870	27/32	.8438
7/8	.0729	.1563	.2396	.3229	.4063	.4896	.5729	.6563	.7396	.8229	.9063	.9896	7/8	.875
29/32	.0755	.1589	.2422	.3255	.4089	.4922	.5755	.6589	.7422	.8255	.9089	.9922	29/32	.9063
15/16	.0781	.1615	.2448	.3281	.4115	.4948	.5781	.6615	.7448	.8281	.9115	.9948	15/16	.9375
31/32	.0807	.1641	.2474	.3307	.4141	.4974	.5807	.6641	.7474	.8307	.9141	.9974	31/32	.9688

TABLE 173

TABLE OF BEVELS

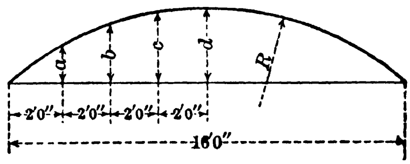
American Bridge Company Standards



Distance a	0		1		2		3		4		5		6		7		8		9		10		11	
	Angle V		Angle V		Angle V		Angle V		Angle V		Angle V		Angle V		Angle V		Angle V		Angle V		Angle V		Angle V	
	Deg.	Min.	Deg.	Min.	Deg.	Min.	Deg.	Min.	Deg.	Min.	Deg.	Min.	Deg.	Min.	Deg.	Min.	Deg.	Min.	Deg.	Min.	Deg.	Min.	Deg.	Min.
0	0	00	4	46	9	28	14	02	18	26	22	37	26	34	30	15	33	41	36	52	39	48	42	31
1/32	0	09	4	55	9	36	14	11	18	34	22	45	26	41	30	22	33	48	36	58	39	54	42	35
1/16	0	18	5	04	9	45	14	19	18	42	22	52	26	48	30	29	33	54	37	04	39	59	42	40
3/32	0	27	5	12	9	54	14	27	18	50	23	00	26	55	30	35	34	00	37	09	40	04	42	45
1/8	0	36	5	21	10	03	14	36	18	58	23	08	27	02	30	42	34	06	37	15	40	09	42	50
5/32	0	45	5	30	10	11	14	44	19	06	23	15	27	10	30	49	34	12	37	21	40	15	42	55
3/16	0	54	5	39	10	20	14	53	19	14	23	23	27	17	30	55	34	18	37	26	40	20	43	00
7/32	1	03	5	48	10	29	15	01	19	22	23	30	27	24	31	02	34	24	37	32	40	25	43	04
1/4	1	12	5	57	10	37	15	09	19	30	23	38	27	31	31	08	34	31	37	38	40	30	43	09
9/32	1	21	6	06	10	46	15	18	19	38	23	45	27	38	31	15	34	37	37	43	40	35	43	14
5/16	1	30	6	15	10	54	15	26	19	46	23	53	27	45	31	21	34	43	37	49	40	41	43	19
11/32	1	38	6	23	11	03	15	34	19	54	24	00	27	52	31	28	34	49	37	54	40	46	43	23
3/8	1	47	6	32	11	12	15	43	20	02	24	08	27	59	31	34	34	55	38	00	40	51	43	28
13/32	1	56	6	41	11	20	15	51	20	10	24	15	28	06	31	41	35	01	38	05	40	56	43	33
7/16	2	05	6	50	11	29	15	59	20	18	24	23	28	13	31	47	35	07	38	11	41	01	43	38
15/32	2	14	6	59	11	38	16	07	20	26	24	30	28	20	31	54	35	13	38	17	41	06	43	42
1/2	2	23	7	08	11	46	16	16	20	33	24	37	28	27	32	00	35	19	38	22	41	11	43	47
17/32	2	32	7	16	11	55	16	24	20	41	24	45	28	34	32	07	35	25	38	28	41	16	43	52
9/16	2	41	7	25	12	03	16	32	20	49	24	52	28	40	32	13	35	31	38	33	41	21	43	56
19/32	2	50	7	34	12	12	16	40	20	57	25	00	28	47	32	20	35	37	38	39	41	26	44	01
5/8	2	59	7	43	12	20	16	49	21	05	25	07	28	54	32	26	35	42	38	44	41	31	44	05
21/32	3	08	7	52	12	29	16	57	21	12	25	14	29	01	32	32	35	48	38	49	41	36	44	10
11/16	3	17	8	00	12	37	17	05	21	20	25	22	29	08	32	39	35	54	38	55	41	41	44	15
23/32	3	26	8	09	12	46	17	13	21	28	25	29	29	15	32	45	36	00	39	00	41	46	44	19
3/4	3	35	8	18	12	54	17	21	21	36	25	36	29	21	32	51	36	06	39	06	41	51	44	24
25/32	3	44	8	27	13	03	17	29	21	43	25	43	29	28	32	58	36	12	39	11	41	56	44	28
13/16	3	52	8	35	13	11	17	38	21	51	25	51	29	35	33	04	36	18	39	16	42	01	44	33
27/32	4	01	8	44	13	20	17	46	21	59	25	58	29	42	33	10	36	23	39	22	42	06	44	37
7/8	4	10	8	53	13	28	17	54	22	07	26	05	29	49	33	17	36	29	39	27	42	11	44	42
29/32	4	19	9	02	13	37	18	02	22	14	26	12	29	55	33	23	36	35	39	32	42	16	44	47
15/16	4	28	9	10	13	45	18	10	22	22	26	20	30	02	33	29	36	41	39	38	42	21	44	51
31/32	4	37	9	19	13	54	18	18	22	30	26	27	30	09	33	35	36	46	39	43	42	26	44	56

TABLE 174
ORDINATES FOR 16′–0″ CHORDS
American Bridge Company Standards

On all drawings for curved work where radius exceeds facilities of Templet Shop Floor, make a sketch as shown giving ordinates from table.



Radius R Ft. In.	Ordinates for 16′–0″ Templet in Inches				Radius R Ft. In.	Ordinates for 16′–0″ Templet in Inches				Radius R Ft. In.	Ordinates for 16′–0″ Templet in Inches			
	a	b	c	d		a	b	c	d		a	b	c	d
16′–6″	11 1/4	18 7/8	23 3/8	24 7/8	24′–8″	7 1/8	12 1/8	15	16	51′–6″	3 1/4	5 5/8	7	7 1/2
16–8	11 1/8	18 3/4	23 1/8	24 1/2	25–0	7	11 7/8	14 7/8	15 3/4	53–0	3 1/4	5 1/2	6 7/8	7 1/4
16–10	11	18 1/2	22 7/8	24 1/4	25–4	6 7/8	11 3/4	14 5/8	15 1/2	54–6	3 1/8	5 3/8	6 5/8	7 1/8
17–0	10 7/8	18 1/4	22 5/8	24	25–8	6 3/4	11 5/8	14 3/8	15 3/8	56–0	3	5 1/8	6 1/2	6 7/8
17–2	10 3/4	18 1/8	22 3/8	23 3/4	26–0	6 3/4	11 3/8	14 1/4	15 1/8	58–0	2 7/8	5	6 1/4	6 5/8
17–4	10 5/8	17 7/8	22 1/8	23 1/2	26–4	6 5/8	11 1/4	14	14 7/8	60–0	2 7/8	4 7/8	6	6 3/8
17–6	10 1/2	17 5/8	21 7/8	23 1/4	26–8	6 1/2	11 1/8	13 7/8	14 3/4	62–6	2 3/4	4 5/8	5 3/4	6 1/8
17–8	10 3/8	17 1/2	21 5/8	23	27–0	6 1/2	11	13 5/8	14 1/2	65–0	2 5/8	4 1/2	5 1/2	5 7/8
17–10	10 1/4	17 1/4	21 3/8	22 3/4	27–6	6 3/8	10 3/4	13 3/8	14 1/4	67–6	2 1/2	4 1/4	5 3/8	5 3/4
18–0	10 1/8	17 1/8	21 1/8	22 1/2	28–0	6 1/4	10 1/2	13 1/8	14	70–0	2 3/8	4 1/8	5 1/8	5 1/2
18–2	10	16 7/8	21	22 1/4	28–6	6 1/8	10 3/8	12 7/8	13 3/4	72–6	2 3/8	4	5	5 3/8
18–4	9 7/8	16 3/4	20 3/4	22	29–0	6	10 1/8	12 5/8	13 1/2	75–0	2 1/4	3 7/8	4 7/8	5 1/8
18–6	9 7/8	16 5/8	20 1/2	21 7/8	29–6	5 7/8	10	12 1/2	13 1/4	77–6	2 1/8	3 3/4	4 5/8	5
18–8	9 3/4	16 3/8	20 3/8	21 5/8	30–0	5 3/4	9 7/8	12 1/4	13	80–0	2 1/8	3 5/8	4 1/2	4 3/4
18–10	9 5/8	16 1/4	20 1/8	21 3/8	30–6	5 5/8	9 5/8	12	12 7/8	84–0	2	3 1/2	4 1/4	4 5/8
19–0	9 1/2	16 1/8	19 7/8	21 1/4	31–0	5 5/8	9 1/2	11 7/8	12 5/8	88–0	1 7/8	3 1/4	4 1/8	4 3/8
19–2	9 3/8	15 7/8	19 3/4	21	31–6	5 1/2	9 3/8	11 5/8	12 3/8	92–0	1 7/8	3 1/8	3 7/8	4 1/8
19–4	9 3/8	15 3/4	19 1/2	20 3/4	32–0	5 3/8	9 1/8	11 1/2	12 1/4	96–0	1 3/4	3	3 3/4	4
19–6	9 1/4	15 5/8	19 3/8	20 5/8	32–9	5 1/4	9	11 1/8	11 7/8	100–0	1 5/8	2 7/8	3 5/8	3 7/8
19–8	9 1/8	15 1/2	19 1/8	20 3/8	33–6	5 1/8	8 3/4	10 7/8	11 5/8	105–0	1 5/8	2 3/4	3 3/8	3 5/8
19–10	9 1/8	15 3/8	19	20 1/4	34–3	5	8 1/2	10 5/8	11 3/8	110–0	1 1/2	2 5/8	3 1/4	3 1/2
20–0	9	15 1/8	18 7/8	20	35–0	4 7/8	8 3/8	10 3/8	11 1/8	115–0	1 1/2	2 1/2	3 1/8	3 3/8
20–3	8 7/8	15	18 5/8	19 3/4	35–9	4 3/4	8 1/8	10 1/4	10 7/8	120–0	1 3/8	2 3/8	3	3 1/4
20–6	8 3/4	14 3/4	18 3/8	19 1/2	36–6	4 3/4	8	10	10 5/8	130–0	1 1/4	2 1/4	2 3/4	3
20–9	8 5/8	14 5/8	18 1/8	19 1/4	37–3	4 5/8	7 7/8	9 3/4	10 3/8	140–0	1 1/4	2	2 5/8	2 3/4
21–0	8 1/2	14 3/8	17 7/8	19	38–0	4 1/2	7 5/8	9 5/8	10 1/4	150–0	1 1/8	1 7/8	2 3/8	2 1/2
21–3	8 3/8	14 1/4	17 5/8	18 3/4	38–9	4 3/8	7 1/2	9 3/8	10	160–0	1	1 3/4	2 1/4	2 3/8
21–6	8 1/4	14	17 3/8	18 1/2	39–6	4 3/8	7 3/8	9 1/4	9 7/8	180–0	7/8	1 5/8	2	2 1/8
21–9	8 1/8	13 7/8	17 1/4	18 1/4	40–3	4 1/4	7 1/4	9	9 5/8	200–0	7/8	1 1/2	1 3/4	1 7/8
22–0	8 1/8	13 5/8	17	18 1/8	41–0	4 1/8	7 1/8	8 7/8	9 1/2	225–0	3/4	1 1/4	1 5/8	1 3/4
22–3	8	13 1/2	16 3/4	17 7/8	42–0	4	6 7/8	8 5/8	9 1/4	250–0	5/8	1 1/8	1 1/2	1 1/2
22–6	7 7/8	13 3/8	16 5/8	17 5/8	43–0	4	6 3/4	8 1/2	9	300–0	1/2	1	1 1/4	1 1/4
22–9	7 3/4	13 1/8	16 3/8	17 3/8	44–0	3 7/8	6 5/8	8 1/4	8 3/4	350–0	1/2	7/8	1	1 1/8
23–0	7 5/8	13	16 1/4	17 1/4	45–0	3 3/4	6 1/2	8 1/8	8 5/8	400–0	3/8	3/4	7/8	1
23–4	7 1/2	12 7/8	16	17	46–3	3 5/8	6 1/4	7 7/8	8 3/8	500–0	3/8	5/8	3/4	3/4
23–8	7 1/2	12 5/8	15 3/4	16 3/4	47–6	3 5/8	6 1/8	7 5/8	8 1/8	625–0	1/4	1/2	5/8	5/8
24–0	7 3/8	12 1/2	15 1/2	16 1/2	48–9	3 1/2	6	7 1/2	7 7/8	750–0	1/4	3/8	1/2	1/2
24–4	7 1/4	12 1/4	15 1/4	16 1/4	50–0	3 3/8	5 3/4	7 1/4	7 3/4	1000–0	1/8	1/4	3/8	3/8

TABLE 175

Natural Tangents

Degrees	0′	5′	10′	15′	20′	25′	30′	35′	40′	45′	50′	55′	60′	Degrees
0	.0000	.0015	.0029	.0044	.0058	.0073	.0087	.0102	.0116	.0131	.0146	.0160	.0175	0
1	.0175	.0189	.0204	.0218	.0233	.0247	.0262	.0276	.0291	.0306	.0320	.0335	.0349	1
2	.0349	.0364	.0378	.0393	.0407	.0422	.0437	.0451	.0466	.0480	.0495	.0509	.0524	2
3	.0524	.0539	.0553	.0568	.0582	.0597	.0612	.0626	.0641	.0655	.0670	.0685	.0699	3
4	.0699	.0714	.0729	.0743	.0758	.0772	.0787	.0802	.0816	.0831	.0846	.0860	.0875	4
5	.0875	.0890	.0904	.0919	.0934	.0948	.0963	.0978	.0992	.1007	.1022	.1036	.1051	5
6	.1051	.1066	.1080	.1095	.1110	.1125	.1139	.1154	.1169	.1184	.1198	.1213	.1228	6
7	.1228	.1243	.1257	.1272	.1287	.1302	.1317	.1331	.1346	.1361	.1376	.1391	.1405	7
8	.1405	.1420	.1435	.1450	.1465	.1480	.1495	.1509	.1524	.1539	.1554	.1569	.1584	8
9	.1584	.1599	.1614	.1629	.1644	.1658	.1673	.1688	.1703	.1718	.1733	.1748	.1763	9
10	.1763	.1778	.1793	.1808	.1823	.1838	.1853	.1868	.1883	.1899	.1914	.1929	.1944	10
11	.1944	.1959	.1974	.1989	.2004	.2019	.2035	.2050	.2065	.2080	.2095	.2110	.2126	11
12	.2126	.2141	.2156	.2171	.2186	.2202	.2217	.2232	.2247	.2263	.2278	.2293	.2309	12
13	.2309	.2324	.2339	.2355	.2370	.2385	.2401	.2416	.2432	.2447	.2462	.2478	.2493	13
14	.2493	.2509	.2524	.2540	.2555	.2571	.2586	.2602	.2617	.2633	.2648	.2664	.2679	14
15	.2679	.2695	.2711	.2726	.2742	.2758	.2773	.2789	.2805	.2820	.2836	.2852	.2867	15
16	.2867	.2883	.2899	.2915	.2931	.2946	.2962	.2978	.2994	.3010	.3026	.3041	.3057	16
17	.3057	.3073	.3089	.3105	.3121	.3137	.3153	.3169	.3185	.3201	.3217	.3233	.3249	17
18	.3249	.3265	.3281	.3298	.3314	.3330	.3346	.3362	.3378	.3395	.3411	.3427	.3443	18
19	.3443	.3460	.3476	.3492	.3508	.3525	.3541	.3558	.3574	.3590	.3607	.3623	.3640	19
20	.3640	.3656	.3673	.3689	.3706	.3722	.3739	.3755	.3772	.3789	.3805	.3822	.3839	20
21	.3839	.3855	.3872	.3889	.3906	.3922	.3939	.3956	.3973	.3990	.4006	.4023	.4040	21
22	.4040	.4057	.4074	.4091	.4108	.4125	.4142	.4159	.4176	.4193	.4210	.4228	.4245	22
23	.4245	.4262	.4279	.4296	.4314	.4331	.4348	.4365	.4383	.4400	.4417	.4435	.4452	23
24	.4452	.4470	.4487	.4505	.4522	.4540	.4557	.4575	.4592	.4610	.4628	.4645	.4663	24
25	.4663	.4681	.4699	.4716	.4734	.4752	.4770	.4788	.4806	.4823	.4841	.4859	.4877	25
26	.4877	.4895	.4913	.4931	.4950	.4968	.4986	.5004	.5022	.5040	.5059	.5077	.5095	26
27	.5095	.5114	.5132	.5150	.5169	.5187	.5206	.5224	.5243	.5261	.5280	.5298	.5317	27
28	.5317	.5336	.5354	.5373	.5392	.5411	.5430	.5448	.5467	.5486	.5505	.5524	.5543	28
29	.5543	.5562	.5581	.5600	.5619	.5639	.5658	.5677	.5696	.5715	.5735	.5754	.5774	29
30	.5774	.5793	.5812	.5832	.5851	.5871	.5890	.5910	.5930	.5949	.5969	.5989	.6009	30
31	.6009	.6028	.6048	.6068	.6088	.6108	.6128	.6148	.6168	.6188	.6208	.6228	.6249	31
32	.6249	.6269	.6289	.6310	.6330	.6350	.6371	.6391	.6412	.6432	.6453	.6473	.6494	32
33	.6494	.6515	.6536	.6556	.6577	.6598	.6619	.6640	.6661	.6682	.6703	.6724	.6745	33
34	.6745	.6766	.6787	.6809	.6830	.6851	.6873	.6894	.6916	.6937	.6959	.6980	.7002	34
35	.7002	.7024	.7046	.7067	.7089	.7111	.7133	.7155	.7177	.7199	.7221	.7243	.7265	35
36	.7265	.7288	.7310	.7332	.7355	.7377	.7400	.7422	.7445	.7467	.7490	.7513	.7536	36
37	.7536	.7558	.7581	.7604	.7627	.7650	.7673	.7696	.7720	.7743	.7766	.7789	.7813	37
38	.7813	.7836	.7860	.7883	.7907	.7931	.7954	.7978	.8002	.8026	.8050	.8074	.8098	38
39	.8098	.8122	.8146	.8170	.8195	.8219	.8243	.8268	.8292	.8317	.8342	.8366	.8391	39
40	.8391	.8416	.8441	.8466	.8491	.8516	.8541	.8566	.8591	.8617	.8642	.8667	.8693	40
41	.8693	.8718	.8744	.8770	.8796	.8821	.8847	.8873	.8899	.8925	.8952	.8978	.9004	41
42	.9004	.9030	.9057	.9083	.9110	.9137	.9163	.9190	.9217	.9244	.9271	.9298	.9325	42
43	.9325	.9352	.9380	.9407	.9435	.9462	.9490	.9517	.9545	.9573	.9601	.9629	.9657	43
44	.9657	.9685	.9713	.9742	.9770	.9798	.9827	.9856	.9884	.9913	.9942	.9971	1.0000	44
Degrees	0′	5′	10′	15′	20′	25′	30′	35′	40′	45′	50′	55′	60′	Degrees

TABLE 176

Squares, Cubes, Square Roots and Cube Roots of Numbers from 1 to 99.

No.	Square.	Cube.	Sq. Root.	Cu. Root.	No.	Square	Cube.	Sq. Root.	Cu. Root.
1	1	1	1.0000	1.0000	50	2500	125000	7.0711	3.6840
2	4	8	1.4142	1.2599	51	2601	132651	7.1414	3.7084
3	9	27	1.7321	1.4422	52	2704	140608	7.2111	3.7325
4	16	64	2.0000	1.5874	53	2809	148877	7.2801	3.7563
5	25	125	2.2361	1.7100	54	2916	157464	7.3485	3.7798
6	36	216	2.4495	1.8171	55	3025	166375	7.4162	3.8030
7	49	343	2.6458	1.9129	56	3136	175616	7.4833	3.8259
8	64	512	2.8284	2.0000	57	3249	185193	7.5498	3.8485
9	81	729	3.0000	2.0801	58	3364	195112	7.6158	3.8709
10	100	1000	3.1623	2.1544	59	3481	205379	7.6811	3.8930
11	121	1331	3.3166	2.2240	60	3600	216000	7.7460	3.9149
12	144	1728	3.4641	2.2894	61	3721	226981	7.8102	3.9365
13	169	2197	3.6056	2.3513	62	3844	238328	7.8740	3.9579
14	196	2744	3.7417	2.4101	63	3969	250047	7.9373	3.9791
15	225	3375	3.8730	2.4662	64	4096	262144	8.0000	4.0000
16	256	4096	4.0000	2.5198	65	4225	274625	8.0623	4.0207
17	289	4913	4.1231	2.5713	66	4356	287496	8.1240	4.0412
18	324	5832	4.2426	2.6207	67	4489	300763	8.1854	4.0615
19	361	6859	4.3589	2.6684	68	4624	314432	8.2462	4.0817
20	400	8000	4.4721	2.7144	69	4761	328509	8.3066	4.1016
21	441	9261	4.5826	2.7589	70	4900	343000	8.3666	4.1213
22	484	10648	4.6904	2.8020	71	5041	357911	8.4261	4.1408
23	529	12167	4.7958	2.8439	72	5184	373248	8.4853	4.1602
24	576	13824	4.8990	2.8845	73	5329	389017	8.5440	4.1793
25	625	15625	5.0000	2.9240	74	5476	405224	8.6023	4.1983
26	676	17576	5.0990	2.9625	75	5625	421875	8.6603	4.2172
27	729	19683	5.1962	3.0000	76	5776	438976	8.7178	4.2358
28	784	21952	5.2915	3.0366	77	5929	456533	8.7750	4.2543
29	841	24389	5.3852	3.0723	78	6084	474552	8.8318	4.2727
30	900	27000	5.4772	3.1072	79	6241	493039	8.8882	4.2908
31	961	29791	5.5678	3.1414	80	6400	512000	8.9443	4.3089
32	1024	32768	5.6569	3.1748	81	6561	531441	9.0000	4.3267
33	1089	35937	5.7446	3.2075	82	6724	551368	9.0554	4.3445
34	1156	39304	5.8310	3.2396	83	6889	571787	9.1104	4.3621
35	1225	42875	5.9161	3.2711	84	7056	592704	9.1652	4.3795
36	1296	46656	6.0000	3.3019	85	7225	614125	9.2195	4.3968
37	1369	50653	6.0828	3.3322	86	7396	636056	9.2736	4.4140
38	1444	54872	6.1644	3.3620	87	7569	658503	9.3274	4.4310
39	1521	59319	6.2450	3.3912	88	7744	681472	9.3808	4.4480
40	1600	64000	6.3246	3.4200	89	7921	704969	9.4340	4.4647
41	1681	68921	6.4031	3.4482	90	8100	729000	9.4868	4.4814
42	1764	74088	6.4807	3.4760	91	8281	753571	9.5394	4.4979
43	1849	79507	6.5574	3.5034	92	8464	778688	9.5917	4.5144
44	1936	85184	6.6332	3.5303	93	8649	804357	9.6437	4.5307
45	2025	91125	6.7082	3.5569	94	8836	830584	9.6954	4.5468
46	2116	97336	6.7823	3.5830	95	9025	857375	9.7468	4.5629
47	2209	103823	6.8557	3.6088	96	9216	884736	9.7980	4.5789
48	2304	110592	6.9282	3.6342	97	9409	912673	9.8489	4.5947
49	2401	117649	7.0000	3.6593	98	9604	941192	9.8995	4.6104
					99	9801	970299	9.9499	4.6261

TABLE 176.—*Continued.*

Squares, Cubes, Square Roots and Cube Roots of Numbers from 100 to 199.

No.	Square.	Cube.	Sq. Root.	Cu. Root.	No.	Square.	Cube.	Sq. Root.	Cu. Root.
100	10000	1000000	10.0000	4.6416	150	22500	3375000	12.2474	5.3133
101	10201	1030301	10.0499	4.6570	151	22801	3442951	12.2882	5.3251
102	10404	1061208	10.0995	4.6723	152	23104	3511808	12.3288	5.3368
103	10609	1092727	10.1489	4.6875	153	23409	3581577	12.3693	5.3485
104	10816	1124864	10.1980	4.7027	154	23716	3652264	12.4097	5.3601
105	11025	1157625	10.2470	4.7177	155	24025	3723875	12.4499	5.3717
106	11236	1191016	10.2956	4.7326	156	24336	3796416	12.4900	5.3832
107	11449	1225043	10.3441	4.7475	157	24649	3869893	12.5300	5.3947
108	11664	1259712	10.3923	4.7622	158	24964	3944312	12.5698	5.4061
109	11881	1295029	10.4403	4.7769	159	25281	4019679	12.6095	5.4175
110	12100	1331000	10.4881	4.7914	160	25600	4096000	12.6491	5.4288
111	12321	1367631	10.5357	4.8059	161	25921	4173281	12.6886	5.4401
112	12544	1404928	10.5830	4.8203	162	26244	4251528	12.7279	5.4514
113	12769	1442897	10.6301	4.8346	163	26569	4330747	12.7671	5.4626
114	12996	1481544	10.6771	4.8488	164	26896	4410944	12.8062	5.4737
115	13225	1520875	10.7238	4.8629	165	27225	4492125	12.8452	5.4848
116	13456	1560896	10.7703	4.8770	166	27556	4574296	12.8841	5.4959
117	13689	1601613	10.8167	4.8910	167	27889	4657463	12.9228	5.5069
118	13924	1643032	10.8628	4.9049	168	28224	4741632	12.9615	5.5178
119	14161	1685159	10.9087	4.9187	169	28561	4826809	13.0000	5.5288
120	14400	1728000	10.9545	4.9324	170	28900	4913000	13.0384	5.5397
121	14641	1771561	11.0000	4.9461	171	29241	5000211	13.0767	5.5505
122	14884	1815848	11.0454	4.9597	172	29584	5088448	13.1149	5.5613
123	15129	1860867	11.0905	4.9732	173	29929	5177717	13.1529	5.5721
124	15376	1906624	11.1355	4.9866	174	30276	5268024	13.1909	5.5828
125	15625	1953125	11.1803	5.0000	175	30625	5359375	13.2288	5.5934
126	15876	2000376	11.2250	5.0133	176	30976	5451776	13.2665	5.6041
127	16129	2048383	11.2694	5.0265	177	31329	5545233	13.3041	5.6147
128	16384	2097152	11.3137	5.0397	178	31684	5639752	13.3417	5.6252
129	16641	2146689	11.3578	5.0528	179	32041	5735339	13.3791	5.6357
130	16900	2197000	11.4018	5.0658	180	32400	5832000	13.4164	5.6462
131	17161	2248091	11.4455	5.0788	181	32761	5929741	13.4536	5.6567
132	17424	2299968	11.4891	5.0916	182	33124	6028568	13.4907	5.6671
133	17689	2352637	11.5326	5.1045	183	33489	6128487	13.5277	5.6774
134	17956	2406104	11.5758	5.1172	184	33856	6229504	13.5647	5.6877
135	18225	2460375	11.6190	5.1299	185	34225	6331625	13.6015	5.6980
136	18496	2515456	11.6619	5.1426	186	34596	6434856	13.6382	5.7083
137	18769	2571353	11.7047	5.1551	187	34969	6539203	13.6748	5.7185
138	19044	2628072	11.7473	5.1676	188	35344	6644672	13.7113	5.7287
139	19321	2685619	11.7898	5.1801	189	35721	6751269	13.7477	5.7388
140	19600	2744000	11.8322	5.1925	190	36100	6859000	13.7840	5.7489
141	19881	2803221	11.8743	5.2048	191	36481	6967871	13.8203	5.7590
142	20164	2863288	11.9164	5.2171	192	36864	7077888	13.8564	5.7690
143	20449	2924207	11.9583	5.2293	193	37249	7189057	13.8924	5.7790
144	20736	2985984	12.0000	5.2415	194	37636	7301384	13.9284	5.7890
145	21025	3048625	12.0416	5.2536	195	38025	7414875	13.9642	5.7989
146	21316	3112136	12.0830	5.2656	196	38416	7529536	14.0000	5.8088
147	21609	3176523	12.1244	5.2776	197	38809	7645373	14.0357	5.8186
148	21904	3241792	12.1655	5.2896	198	39204	7762392	14.0712	5.8285
149	22201	3307949	12.2066	5.3015	199	39601	7880599	14.1067	5.8383

TABLE 176.—*Continued.*

SQUARES, CUBES, SQUARE ROOTS AND CUBE ROOTS OF NUMBERS FROM 200 TO 299.

No.	Square.	Cube.	Sq. Root.	Cu. Root.	No.	Square.	Cube.	Sq. Root.	Cu. Root.
200	40000	8000000	14.1421	5.8480	250	62500	15625000	15.8114	6.2996
201	40401	8120601	14.1774	5.8578	251	63001	15813251	15.8430	6.3080
202	40804	8242408	14.2127	5.8675	252	63504	16003008	15.8745	6.3164
203	41209	8365427	14.2478	5.8771	253	64009	16194277	15.9060	6.3247
204	41616	8489664	14.2829	5.8868	254	64516	16387064	15.9374	6.3330
205	42025	8615125	14.3178	5.8964	255	65025	16581375	15.9687	6.3413
206	42436	8741816	14.3527	5.9059	256	65536	16777216	16.0000	6.3496
207	42849	8869743	14.3875	5.9155	257	66049	16974593	16.0312	6.3579
208	43264	8998912	14.4222	5.9250	258	66564	17173512	16.0624	6.3661
209	43681	9129329	14.4568	5.9345	259	67081	17373979	16.0935	6.3743
210	44100	9261000	14.4914	5.9439	260	67600	17576000	16.1245	6.3825
211	44521	9393931	14.5258	5.9533	261	68121	17779581	16.1555	6.3907
212	44944	9528128	14.5602	5.9627	262	68644	17984728	16.1864	6.3988
213	45369	9663597	14.5945	5.9721	263	69169	18191447	16.2173	6.4070
214	45796	9800344	14.6287	5.9814	264	69696	18399744	16.2481	6.4151
215	46225	9938375	14.6629	5.9907	265	70225	18609625	16.2788	6.4232
216	46656	10077696	14.6969	6.0000	266	70756	18821096	16.3095	6.4312
217	47089	10218313	14.7309	6.0092	267	71289	19034163	16.3401	6.4393
218	47524	10360232	14.7648	6.0185	268	71824	19248832	16.3707	6.4473
219	47961	10503459	14.7986	6.0277	269	72361	19465109	16.4012	6.4553
220	48400	10648000	14.8324	6.0368	270	72900	19683000	16.4317	6.4633
221	48841	10793861	14.8661	6.0459	271	73441	19902511	16.4621	6.4713
222	49284	10941048	14.8997	6.0550	272	73984	20123648	16.4924	6.4792
223	49729	11089567	14.9332	6.0641	273	74529	20346417	16.5227	6.4872
224	50176	11239424	14.9666	6.0732	274	75076	20570824	16.5529	6.4951
225	50625	11390625	15.0000	6.0822	275	75625	20796875	16.5831	6.5030
226	51076	11543176	15.0333	6.0912	276	76176	21024576	16.6132	6.5108
227	51529	11697083	15.0665	6.1002	277	76729	21253933	16.6433	6.5187
228	51984	11852352	15.0997	6.1091	278	77284	21484952	16.6733	6.5265
229	52441	12008989	15.1327	6.1180	279	77841	21717639	16.7033	6.5343
230	52900	12167000	15.1658	6.1269	280	78400	21952000	16.7332	6.5421
231	53361	12326391	15.1987	6.1358	281	78961	22188041	16.7631	6.5499
232	53824	12487168	15.2315	6.1446	282	79524	22425768	16.7929	6.5577
233	54289	12649337	15.2643	6.1534	283	80089	22665187	16.8226	6.5654
234	54756	12812904	15.2971	6.1622	284	80656	22906304	16.8523	6.5731
235	55225	12977875	15.3297	6.1710	285	81225	23149125	16.8819	6.5808
236	55696	13144256	15.3623	6.1797	286	81796	23393656	16.9115	6.5885
237	56169	13312053	15.3948	6.1885	287	82369	23639903	16.9411	6.5962
238	56644	13481272	15.4272	6.1972	288	82944	23887872	16.9706	6.6039
239	57121	13651919	15.4596	6.2058	289	83521	24137569	17.0000	6.6115
240	57600	13824000	15.4919	6.2145	290	84100	24389000	17.0294	6.6191
241	58081	13997521	15.5242	6.2231	291	84681	24642171	17.0587	6.6267
242	58564	14172488	15.5563	6.2317	292	85264	24897088	17.0880	6.6343
243	59049	14348907	15.5885	6.2403	293	85849	25153757	17.1172	6.6419
244	59536	14526784	15.6205	6.2488	294	86436	25412184	17.1464	6.6494
245	60025	14706125	15.6525	6.2573	295	87025	25672375	17.1756	6.6569
246	60516	14886936	15.6844	6.2658	296	87616	25934336	17.2047	6.6644
247	61009	15069223	15.7162	6.2743	297	88209	26198073	17.2337	6.6719
248	61504	15252992	15.7480	6.2828	298	88804	26463592	17.2627	6.6794
249	62001	15438249	15.7797	6.2912	299	89401	26730899	17.2916	6.6869

TABLE 176.—*Continued.*

Squares, Cubes, Square Roots and Cube Roots of Numbers from 300 to 399.

No.	Square.	Cube.	Sq. Root.	Cu. Root.	No.	Square.	Cube.	Sq. Root.	Cu. Root.
300	90000	27000000	17.3205	6.6943	350	122500	42875000	18.7083	7.0473
301	90601	27270901	17.3494	6.7018	351	123201	43243551	18.7350	7.0540
302	91204	27543608	17.3781	6.7092	352	123904	43614208	18.7617	7.0607
303	91809	27818127	17.4069	6.7166	353	124609	43986977	18.7883	7.0674
304	92416	28094464	17.4356	6.7240	354	125316	44361864	18.8149	7.0740
305	93025	28372625	17.4642	6.7313	355	126025	44738875	18.8414	7.0807
306	93636	28652616	17.4929	6.7387	356	126736	45118016	18.8680	7.0873
307	94249	28934443	17.5214	6.7460	357	127449	45499293	18.8944	7.0940
308	94864	29218112	17.5499	6.7533	358	128164	45882712	18.9209	7.1006
309	95481	29503629	17.5784	6.7606	359	128881	46268279	18.9473	7.1072
310	96100	29791000	17.6068	6.7679	360	129600	46656000	18.9737	7.1138
311	96721	30080231	17.6352	6.7752	361	130321	47045881	19.0000	7.1204
312	97344	30371328	17.6635	6.7824	362	131044	47437928	19.0263	7.1269
313	97969	30664297	17.6918	6.7897	363	131769	47832147	19.0526	7.1335
314	98596	30959144	17.7200	6.7969	364	132496	48228544	19.0788	7.1400
315	99225	31255875	17.7482	6.8041	365	133225	48627125	19.1050	7.1466
316	99856	31554496	17.7764	6.8113	366	133956	49027896	19.1311	7.1531
317	100489	31855013	17.8045	6.8185	367	134689	49430863	19.1572	7.1596
318	101124	32157432	17.8326	6.8256	368	135424	49836032	19.1833	7.1661
319	101761	32461759	17.8606	6.8328	369	136161	50243409	19.2094	7.1726
320	102400	32768000	17.8885	6.8399	370	136900	50653000	19.2354	7.1791
321	103041	33076161	17.9165	6.8470	371	137641	51064811	19.2614	7.1855
322	103684	33386248	17.9444	6.8541	372	138384	51478848	19.2873	7.1920
323	104329	33698267	17.9722	6.8612	373	139129	51895117	19.3132	7.1984
324	104976	34012224	18.0000	6.8683	374	139876	52313624	19.3391	7.2048
325	105625	34328125	18.0278	6.8753	375	140625	52734375	19.3649	7.2112
326	106276	34645976	18.0555	6.8824	376	141376	53157376	19.3907	7.2177
327	106929	34965783	18.0831	6.8894	377	142129	53582633	19.4165	7.2240
328	107584	35287552	18.1108	6.8964	378	142884	54010152	19.4422	7.2304
329	108241	35611289	18.1384	6.9034	379	143641	54439939	19.4679	7.2368
330	108900	35937000	18.1659	6.9104	380	144400	54872000	19.4936	7.2432
331	109561	36264691	18.1934	6.9174	381	145161	55306341	19.5192	7.2495
332	110224	36594368	18.2209	6.9244	382	145924	55742968	19.5448	7.2558
333	110889	36926037	18.2483	6.9313	383	146689	56181887	19.5704	7.2622
334	111556	37259704	18.2757	6.9382	384	147456	56623104	19.5959	7.2685
335	112225	37595375	18.3030	6.9451	385	148225	57066625	19.6214	7.2748
336	112896	37933056	18.3303	6.9521	386	148996	57512456	19.6469	7.2811
337	113569	38272753	18.3576	6.9589	387	149769	57960603	19.6723	7.2874
338	114244	38614472	18.3848	6.9658	388	150544	58411072	19.6977	7.2936
339	114921	38958219	18.4120	6.9727	389	151321	58863869	19.7231	7.2999
340	115600	39304000	18.4391	6.9795	390	152100	59319000	19.7484	7.3061
341	116281	39651821	18.1662	6.9864	391	152881	59776471	19.7737	7.3124
342	116964	40001688	18.4932	6.9932	392	153664	60236288	19.7990	7.3186
343	117649	40353607	18.5203	7.0000	393	154449	60698457	19.8242	7.3248
344	118336	40707584	18.5472	7.0068	394	155236	61162984	19.8494	7.3310
345	119025	41063625	18.5742	7.0136	395	156025	61629875	19.8746	7.3372
346	119716	41421736	18.6011	7.0203	396	156816	62099136	19.8997	7.3434
347	120409	41781923	18.6279	7.0271	397	157609	62570773	19.9249	7.3496
348	121104	42144192	18.6548	7.0338	398	158404	63044792	19.9499	7.3558
349	121801	42508549	18.6815	7.0406	399	159201	63521199	19.9750	7.3619

TABLE 176.—*Continued.*

Squares, Cubes, Square Roots and Cube Roots of Numbers from 400 to 499.

No.	Square.	Cube.	Sq. Root.	Cu. Root.	No.	Square.	Cube.	Sq. Root.	Cu. Root.
400	160000	64000000	20.0000	7.3681	450	202500	91125000	21.2132	7.6631
401	160801	64481201	20.0250	7.3742	451	203401	91733851	21.2368	7.6688
402	161604	64964808	20.0499	7.3803	452	204304	92345408	21.2603	7.6744
403	162409	65450827	20.0749	7.3864	453	205209	92959677	21.2838	7.6801
404	163216	65939264	20.0998	7.3925	454	206116	93576664	21.3073	7.6857
405	164025	66430125	20.1246	7.3986	455	207025	94196375	21.3307	7.6914
406	164836	66923416	20.1494	7.4047	456	207936	94818816	21.3542	7.6970
407	165649	67419143	20.1742	7.4108	457	208849	95443993	21.3776	7.7026
408	166464	67917312	20.1990	7.4169	458	209764	96071912	21.4009	7.7082
409	167281	68417929	20.2237	7.4229	459	210681	96702579	21.4243	7.7138
410	168100	68921000	20.2485	7.4290	460	211600	97336000	21.4476	7.7194
411	168921	69426531	20.2731	7.4350	461	212521	97972181	21.4709	7.7250
412	169744	69934528	20.2978	7.4410	462	213444	98611128	21.4942	7.7306
413	170569	70444997	20.3224	7.4470	463	214369	99252847	21.5174	7.7362
414	171396	70957944	20.3470	7.4530	464	215296	99897344	21.5407	7.7418
415	172225	71473375	20.3715	7.4590	465	216225	100544625	21.5639	7.7473
416	173056	71991296	20.3961	7.4650	466	217156	101194696	21.5870	7.7529
417	173889	72511713	20.4206	7.4710	467	218089	101847563	21.6102	7.7584
418	174724	73034632	20.4450	7.4770	468	219024	102503232	21.6333	7.7639
419	175561	73560059	20.4695	7.4829	469	219961	103161709	21.6564	7.7695
420	176400	74088000	20.4939	7.4889	470	220900	103823000	21.6795	7.7750
421	177241	74618461	20.5183	7.4948	471	221841	104487111	21.7025	7.7805
422	178084	75151448	20.5426	7.5007	472	222784	105154048	21.7256	7.7860
423	178929	75686967	20.5670	7.5067	473	223729	105823817	21.7486	7.7915
424	179776	76225024	20.5913	7.5126	474	224676	106496424	21.7715	7.7970
425	180625	76765625	20.6155	7.5185	475	225625	107171875	21.7945	7.8025
426	181476	77308776	20.6398	7.5244	476	226576	107850176	21.8174	7.8079
427	182329	77854483	20.6640	7.5302	477	227529	108531333	21.8403	7.8134
428	183184	78402752	20.6882	7.5361	478	228484	109215352	21.8632	7.8188
429	184041	78953589	20.7123	7.5420	479	229441	109902239	21.8861	7.8243
430	184900	79507000	20.7364	7.5478	480	230400	110592000	21.9089	7.8297
431	185761	80062991	20.7605	7.5537	481	231361	111284641	21.9317	7.8352
432	186624	80621568	20.7846	7.5595	482	232324	111980168	21.9545	7.8406
433	187489	81182737	20.8087	7.5654	483	233289	112678587	21.9773	7.8460
434	188356	81746504	20.8327	7.5712	484	234256	113379904	22.0000	7.8514
435	189225	82312875	20.8567	7.5770	485	235225	114084125	22.0227	7.8568
436	190096	82881856	20.8806	7.5828	486	236196	114791256	22.0454	7.8622
437	190969	83453453	20.9045	7.5886	487	237169	115501303	22.0681	7.8676
438	191844	84027672	20.9284	7.5944	488	238144	116214272	22.0907	7.8730
439	192721	84604519	20.9523	7.6001	489	239121	116930169	22.1133	7.8784
440	193600	85184000	20.9762	7.6059	490	240100	117649000	22.1359	7.8837
441	194481	85766121	21.0000	7.6117	491	241081	118370771	22.1585	7.8891
442	195364	86350888	21.0238	7.6174	492	242064	119095488	22.1811	7.8944
443	196249	86938307	21.0476	7.6232	493	243049	119823157	22.2036	7.8998
444	197136	87528384	21.0713	7.6289	494	244036	120553784	22.2261	7.9051
445	198025	88121125	21.0950	7.6346	495	245025	121287375	22.2486	7.9105
446	198916	88716536	21.1187	7.6403	496	246016	122023936	22.2711	7.9158
447	199809	89314623	21.1424	7.6460	497	247009	122763473	22.2935	7.9211
448	200704	89915392	21.1660	7.6517	498	248004	123505992	22.3159	7.9264
449	201601	90518849	21.1896	7.6574	499	249001	124251499	22.3383	7.9317

TABLE 176.—*Continued.*

Squares, Cubes, Square Roots and Cube Roots of Numbers from 500 to 599.

No.	Square.	Cube.	Sq. Root.	Cu. Root.	No.	Square.	Cube.	Sq. Root.	Cu. Root.
500	250000	125000000	22.3607	7.9370	550	302500	166375000	23.4521	8.1932
501	251001	125751501	22.3830	7.9423	551	303601	167284151	23.4734	8.1982
502	252004	126506008	22.4054	7.9476	552	304704	168196608	23.4947	8.2031
503	253009	127263527	22.4277	7.9528	553	305809	169112377	23.5160	8.2081
504	254016	128024064	22.4499	7.9581	554	306916	170031464	23.5372	8.2130
505	255025	128787625	22.4722	7.9634	555	308025	170953875	23.5584	8.2180
506	256036	129554216	22.4944	7.9686	556	309136	171879616	23.5797	8.2229
507	257049	130323843	22.5167	7.9739	557	310249	172808693	23.6008	8.2278
508	258064	131096512	22.5389	7.9791	558	311364	173741112	23.6220	8.2327
509	259081	131872229	22.5610	7.9843	559	312481	174676879	23.6432	8.2377
510	260100	132651000	22.5832	7.9896	560	313600	175616000	23.6643	8.2426
511	261121	133432831	22.6053	7.9948	561	314721	176558481	23.6854	8.2475
512	262144	134217728	22.6274	8.0000	562	315844	177504328	23.7065	8.2524
513	263169	135005697	22.6495	8.0052	563	316969	178453547	23.7276	8.2573
514	264196	135796744	22.6716	8.0104	564	318096	179406144	23.7487	8.2621
515	265225	136590875	22.6936	8.0156	565	319225	180362125	23.7697	8.2670
516	266256	137388096	22.7156	8.0208	566	320356	181321496	23.7908	8.2719
517	267289	138188413	22.7376	8.0260	567	321489	182284263	23.8118	8.2768
518	268324	138991832	22.7596	8.0311	568	322624	183250432	23.8328	8.2816
519	269361	139798359	22.7816	8.0363	569	323761	184220009	23.8537	8.2865
520	270400	140608000	22.8035	8.0415	570	324900	185193000	23.8747	8.2913
521	271441	141420761	22.8254	8.0466	571	326041	186169411	23.8956	8.2962
522	272484	142236648	22.8473	8.0517	572	327184	187149248	23.9165	8.3010
523	273529	143055667	22.8692	8.0569	573	328329	188132517	23.9374	8.3059
524	274576	143877824	22.8910	8.0620	574	329476	189119224	23.9583	8.3107
525	275625	144703125	22.9129	8.0671	575	330625	190109375	23.9792	8.3155
526	276676	145531576	22.9347	8.0723	576	331776	191102976	24.0000	8.3203
527	277729	146363183	22.9565	8.0774	577	332929	192100033	24.0208	8.3251
528	278784	147197952	22.9783	8.0825	578	334084	193100552	24.0416	8.3300
529	279841	148035889	23.0000	8.0876	579	335241	194104539	24.0624	8.3348
530	280900	148877000	23.0217	8.0927	580	336400	195112000	24.0832	8.3396
531	281961	149721291	23.0434	8.0978	581	337561	196122941	24.1039	8.3443
532	283024	150568768	23.0651	8.1028	582	338724	197137368	24.1247	8.3491
533	284089	151419437	23.0868	8.1079	583	339889	198155287	24.1454	8.3539
534	285156	152273304	23.1084	8.1130	584	341056	199176704	24.1661	8.3587
535	286225	153130375	23.1301	8.1180	585	342225	200201625	24.1868	8.3634
536	287296	153990656	23.1517	8.1231	586	343396	201230056	24.2074	8.3682
537	288369	154854153	23.1733	8.1281	587	344569	202262003	24.2281	8.3730
538	289444	155720872	23.1948	8.1332	588	345744	203297472	24.2487	8.3777
539	290521	156590819	23.2164	8.1382	589	346921	204336469	24.2693	8.3825
540	291600	157464000	23.2379	8.1433	590	348100	205379000	24.2899	8.3872
541	292681	158340421	23.2594	8.1483	591	349281	206425071	24.3105	8.3919
542	293764	159220088	23.2809	8.1533	592	350464	207474688	24.3311	8.3967
543	294849	160103007	23.3024	8.1583	593	351649	208527857	24.3516	8.4014
544	295936	160989184	23.3238	8.1633	594	352836	209584584	24.3721	8.4061
545	297025	161878625	23.3452	8.1683	595	354025	210644875	24.3926	8.4108
546	298116	162771336	23.3666	8.1733	596	355216	211708736	24.4131	8.4155
547	299209	163667323	23.3880	8.1783	597	356409	212776173	24.4336	8.4202
548	300304	164566592	23.4094	8.1833	598	357604	213847192	24.4540	8.4249
549	301401	165469149	23.4307	8.1882	599	358801	214921799	24.4745	8.4296

TABLE 176.—*Continued.*

Squares, Cubes, Square Roots and Cube Roots of Numbers from 600 to 699.

No.	Square.	Cube.	Sq. Root.	Cu. Root.	No.	Square.	Cube.	Sq. Root.	Cu. Root.
600	360000	216000000	24.4949	8.4343	650	422500	274625000	25.4951	8.6624
601	361201	217081801	24.5153	8.4390	651	423801	275894451	25.5147	8.6668
602	362404	218167208	24.5357	8.4437	652	425104	277167808	25.5343	8.6713
603	363609	219256227	24.5561	8.4484	653	426409	278445077	25.5539	8.6757
604	364816	220348864	24.5764	8.4530	654	427716	279726264	25.5734	8.6801
605	366025	221445125	24.5967	8.4577	655	429025	281011375	25.5930	8.6845
606	367236	222545016	24.6171	8.4623	656	430336	282300416	25.6125	8.6890
607	368449	223648543	24.6374	8.4670	657	431649	283593393	25.6320	8.6934
608	369664	224755712	24.6577	8.4716	658	432964	284890312	25.6515	8.6978
609	370881	225866529	24.6779	8.4763	659	434281	286191179	25.6710	8.7022
610	372100	226981000	24.6982	8.4809	660	435600	287496000	25.6905	8.7066
611	373321	228099131	24.7184	8.4856	661	436921	288804781	25.7099	8.7110
612	374544	229220928	24.7386	8.4902	662	438244	290117528	25.7294	8.7154
613	375769	230346397	24.7588	8.4948	663	439569	291434247	25.7488	8.7198
614	376996	231475544	24.7790	8.4994	664	440896	292754944	25.7682	8.7241
615	378225	232608375	24.7992	8.5040	665	442225	294079625	25.7876	8.7285
616	379456	233744896	24.8193	8.5086	666	443556	295408296	25.8070	8.7329
617	380689	234885113	24.8395	8.5132	667	444889	296740963	25.8263	8.7373
618	381924	236029032	24.8596	8.5178	668	446224	298077632	25.8457	8.7416
619	383161	237176659	24.8797	8.5224	669	447561	299418309	25.8650	8.7460
620	384400	238328000	24.8998	8.5270	670	448900	300763000	25.8844	8.7503
621	385641	239483061	24.9199	8.5316	671	450241	302111711	25.9037	8.7547
622	386884	240641848	24.9399	8.5362	672	451584	303464448	25.9230	8.7590
623	388129	241804367	24.9600	8.5408	673	452929	304821217	25.9422	8.7634
624	389376	242970624	24.9800	8.5453	674	454276	306182024	25.9615	8.7677
625	390625	244140625	25.0000	8.5499	675	455625	307546875	25.9808	8.7721
626	391876	245314376	25.0200	8.5544	676	456976	308915776	26.0000	8.7764
627	393129	246491883	25.0400	8.5590	677	458329	310288733	26.0192	8.7807
628	394384	247673152	25.0599	8.5635	678	459684	311665752	26.0384	8.7850
629	395641	248858189	25.0799	8.5681	679	461041	313046839	26.0576	8.7893
630	396900	250047000	25.0998	8.5726	680	462400	314432000	26.0768	8.7937
631	398161	251239591	25.1197	8.5772	681	463761	315821241	26.0960	8.7980
632	399424	252435968	25.1396	8.5817	682	465124	317214568	26.1151	8.8023
633	400689	253636137	25.1595	8.5862	683	466489	318611987	26.1343	8.8066
634	401956	254840104	25.1794	8.5907	684	467856	320013504	26.1534	8.8109
635	403225	256047875	25.1992	8.5952	685	469225	321419125	26.1725	8.8152
636	404496	257259456	25.2190	8.5997	686	470596	322828856	26.1916	8.8194
637	405769	258474853	25.2389	8.6043	687	471969	324242703	26.2107	8.8237
638	407044	259694072	25.2587	8.6088	688	473344	325660672	26.2298	8.8280
639	408321	260917119	25.2784	8.6132	689	474721	327082769	26.2488	8.8323
640	409600	262144000	25.2982	8.6177	690	476100	328509000	26.2679	8.8366
641	410881	263374721	25.3180	8.6222	691	477481	329939371	26.2869	8.8408
642	412164	264609288	25.3377	8.6267	692	478864	331373888	26.3059	8.8451
643	413449	265847707	25.3574	8.6312	693	480249	332812557	26.3249	8.8493
644	414736	267089984	25.3772	8.6357	694	481636	334255384	26.3439	8.8536
645	416025	268336125	25.3969	8.6401	695	483025	335702375	26.3629	8.8578
646	417316	269586136	25.4165	8.6446	696	484416	337153536	26.3818	8.8621
647	418609	270840023	25.4362	8.6490	697	485809	338608873	26.4008	8.8663
648	419904	272097792	25.4558	8.6535	698	487204	340068392	26.4197	8.8706
649	421201	273359449	25.4755	8.6579	699	488601	341532099	26.4386	8.8748

TABLE 176.—*Continued.*

Squares, Cubes, Square Roots and Cube Roots of Numbers from 700 to 799.

No.	Square.	Cube.	Sq. Root.	Cu. Root.	No.	Square.	Cube.	Sq. Root.	Cu. Root.
700	490000	343000000	26.4575	8.8790	750	562500	421875000	27.3861	9.0856
701	491401	344472101	26 4764	8.8833	751	564001	423564751	27.4044	9.0896
702	492804	345948408	26.4953	8.8875	752	565504	425259008	27.4226	9.0937
703	494209	347428927	26.5141	8.8917	753	567009	426957777	27.4408	9.0977
704	495616	348913664	26.5330	8.8959	754	568516	428661064	27.4591	9.1017
705	497025	350402625	26.5518	8.9001	755	570025	430368875	27.4773	9.1057
706	498436	351895816	26.5707	8.9043	756	571536	432081216	27.4955	9.1098
707	499849	353393243	26.5895	8.9085	757	573049	433798093	27.5136	9.1138
708	501264	354894912	26.6083	8.9127	758	574564	435519512	27.5318	9.1178
709	502681	356400829	26.6271	8.9169	759	576081	437245479	27.5500	9.1218
710	504100	357911000	26.6458	8.9211	760	577600	438976000	27.5681	9.1258
711	505521	359425431	26.6646	8.9253	761	579121	440711081	27.5862	9.1298
712	506944	360944128	26.6833	8.9295	762	580644	442450728	27.6043	9.1338
713	508369	362467097	26.7021	8.9337	763	582169	444194947	27.6225	9.1378
714	509796	363994344	26.7208	8.9373	764	583696	445943744	27.6405	9.1418
715	511225	365525875	26.7395	8.9420	765	585225	447697125	27.6586	9.1458
716	512656	367061696	26.7582	8.9462	766	586756	449455096	27.6767	9.1498
717	514089	368601813	26.7769	8.9503	767	588289	451217663	27.6948	9.1537
718	515524	370146232	26.7955	8.9545	768	589824	452984832	27.7128	9.1577
719	516961	371694959	26.8142	8.9587	769	591361	454756609	27.7308	9.1617
720	518400	373248000	26.8328	8.9628	770	592900	456533000	27.7489	9.1657
721	519841	374805361	26.8514	8.9670	771	594441	458314011	27.7669	9.1696
722	521284	376367048	26.8701	8.9711	772	595984	460099648	27.7849	9.1736
723	522729	377933067	26.8887	8.9752	773	597529	461889917	27.8029	9.1775
724	524176	379503424	26.9072	8.9794	774	599076	463684824	27.8209	9.1815
725	525625	381078125	26.9258	8.9835	775	600625	465484375	27.8388	9.1855
726	527076	382657176	26.9444	8.9876	776	602176	467288576	27.8568	9.1894
727	528529	384240583	26.9629	8.9918	777	603729	469097433	27.8747	9.1933
728	529984	385828352	26.9815	8.9959	778	605284	470910952	27.8927	9.1973
729	531441	387420489	27.0000	9.0000	779	606841	472729139	27.9106	9.2012
730	532900	389017000	27.0185	9.0041	780	608400	474552000	27.9285	9.2052
731	534361	390617891	27.0370	9.0082	781	609961	476379541	27.9464	9.2091
732	535824	392223168	27.0555	9 0123	782	611524	478211768	27.9643	9.2130
733	537289	393832837	27.0740	9.0164	783	613089	480048687	27.9821	9.2170
734	538756	395446904	27.0924	9.0205	784	614656	481890304	28.0000	9.2209
735	540225	397065375	27.1109	9.0246	785	616225	483736625	28.0179	9.2248
736	541696	398688256	27.1293	9.0287	786	617796	485587656	28.0357	9.2287
737	543169	400315553	27.1477	9.0328	787	619369	487443403	28.0535	9.2326
738	544644	401947272	27.1662	9.0369	788	620944	489303872	28.0713	9 2365
739	546121	403583419	27.1846	9.0410	789	622521	491169069	28.0891	9.2404
740	547600	405224000	27.2029	9.0450	790	624100	493039000	28.1069	9.2443
741	549081	406869021	27.2213	9.0491	791	625681	494913671	28 1247	9.2482
742	550564	408518488	27 2397	9.0532	792	627264	496793088	28.1425	9.2521
743	552049	410172407	27.2580	9.0572	793	628849	498677257	28.1603	9.2560
744	553536	411830784	27.2764	9.0613	794	630436	500566184	28.1780	9.2599
745	555025	413493625	27.2947	9.0654	795	632025	502459875	28.1957	9.2638
746	556516	415160936	27.3130	9.0694	796	633616	504358336	28.2135	9.2677
747	558009	416832723	27.3313	9.0735	797	635209	506261573	28.2312	9.2716
748	559504	418508992	27.3496	9.0775	798	636804	508169592	28.2489	9.2754
749	561001	420189749	27.3679	9.0816	799	638401	510082399	28.2666	9.2793

TABLE 176.—*Continued.*

Squares, Cubes, Square Roots and Cube Roots of Numbers from 800 to 899.

No.	Square.	Cube.	Sq. Root.	Cu. Root.	No.	Square.	Cube.	Sq. Root.	Cu. Root.
800	640000	512000000	28.2843	9.2832	850	722500	614125000	29.1548	9.4727
801	641601	513922401	28.3019	9.2870	851	724201	616295051	29.1719	9.4764
802	643204	515849608	28.3196	9.2909	852	725904	618470208	29.1890	9.4801
803	644809	517781627	28.3373	9.2948	853	727609	620650477	29.2062	9.4838
804	646416	519718464	28.3549	9.2986	854	729316	622835864	29.2233	9.4875
805	648025	521660125	28.3725	9.3025	855	731025	625026375	29.2404	9.4912
806	649636	523606616	28.3901	9.3063	856	732736	627222016	29.2575	9.4949
807	651249	525557943	28.4077	9.3102	857	734449	629422793	29.2746	9.4986
808	652864	527514112	28.4253	9.3140	858	736164	631628712	29.2916	9.5023
809	654481	529475129	28.4429	9.3179	859	737881	633839779	29.3087	9.5060
810	656100	531441000	28.4605	9.3217	860	739600	636056000	29.3258	9.5097
811	657721	533411731	28.4781	9.3255	861	741321	638277381	29.3428	9.5134
812	659344	535387328	28.4956	9.3294	862	743044	640503928	29.3598	9.5171
813	660969	537367797	28.5132	9.3332	863	744769	642735647	29.3769	9.5207
814	662596	539353144	28.5307	9.3370	864	746496	644972544	29.3939	9.5244
815	664225	541343375	28.5482	9.3408	865	748225	647214625	29.4109	9.5281
816	665856	543338496	28 5657	9.3447	866	749956	649461896	29.4279	9.5317
817	667489	545338513	28.5832	9.3485	867	751689	651714363	29.4449	9.5354
818	669124	547343432	28.6007	9.3523	868	753424	653972032	29.4618	9.5391
819	670761	549353259	28.6182	9.3561	869	755161	656234909	29.4788	9.5427
820	672400	551368000	28.6356	9.3599	870	756900	658503000	29.4958	9.5464
821	674041	553387661	28.6531	9.3637	871	758641	660776311	29.5127	9.5501
822	675684	555412248	28.6705	9.3675	872	760384	663054848	29.5296	9.5537
823	677329	557441767	28.6880	9.3713	873	762129	665338617	29.5466	9.5574
824	678976	559476224	28.7054	9.3751	874	763876	667627624	29.5635	9.5610
825	680625	561515625	28 7228	9.3789	875	765625	669921875	29.5804	9.5647
826	682276	563559976	28.7402	9.3827	876	767376	672221376	29.5973	9.5683
827	683929	565609283	28.7576	9.3865	877	769129	674526133	29.6142	9.5719
828	685584	567663552	28.7750	9.3902	878	770884	676836152	29.6311	9.5756
829	687241	569722789	28.7924	9.3940	879	772641	679151439	29.6479	9.5792
830	688900	571787000	28.8097	9.3978	880	774400	681472000	29.6648	9.5828
831	690561	573856191	28.8271	9.4016	881	776161	683797841	29.6816	9.5865
832	692224	575930368	28.8444	9.4053	882	777924	686128968	29.6985	9.5901
833	693889	578009537	28.8617	9.4091	883	779689	688465387	29.7153	9.5937
834	695556	580093704	28.8791	9.4129	884	781456	690807104	29.7321	9.5973
835	697225	582182875	28.8964	9.4166	885	783225	693154125	29.7489	9.6010
836	698896	584277056	28.9137	9.4204	886	784996	695506456	29.7658	9.6046
837	700569	586376253	28.9310	9.4241	887	786769	697864103	29 7825	9.6082
838	702244	588480472	28.9482	9.4279	888	788544	700227072	29.7993	9.6118
839	703921	590589719	28.9655	9.4316	889	790321	702595369	29.8161	9.6154
840	705600	592704000	28.9828	9.4354	890	792100	704969000	29.8329	9.6190
841	707281	594823321	29.0000	9.4391	891	793881	707347971	29.8496	9.6226
842	708964	596947688	29.0172	9.4429	892	795664	709732288	29.8664	9.6262
843	710649	599077107	29.0345	9.4466	893	797449	712121957	29.8831	9.6298
844	712336	601211584	29.0517	9.4503	894	799236	714516984	29.8998	9.6334
845	714025	603351125	29.0689	9.4541	895	801025	716917375	29.9166	9 6370
846	715716	605495736	29.0861	9.4578	896	802816	719323136	29.9333	9.6406
847	717409	607645423	29.1033	9.4615	897	804609	721734273	29.9500	9.6442
848	719104	609800192	29.1204	9.4652	898	806404	724150792	29.9666	9.6477
849	720801	611960049	29.1376	9.4690	899	808201	726572699	29.9833	9.6513

TABLE 176.—*Continued.*

Squares, Cubes, Square Roots and Cube Roots of Numbers from 900 to 999.

No.	Square.	Cube.	Sq. Root.	Cu. Root.	No.	Square.	Cube.	Sq. Root.	Cu. Root.
900	810000	729000000	30.0000	9.6549	950	902500	857375000	30.8221	9.8305
901	811801	731432701	30.0167	9.6585	951	904401	860085351	30.8383	9.8339
902	813604	733870808	30.0333	9.6620	952	906304	862801408	30.8545	9.8374
903	815409	736314327	30.0500	9.6656	953	908209	865523177	30.8707	9.8408
904	817216	738763264	30.0666	9.6692	954	910116	868250664	30.8869	9.8443
905	819025	741217625	30.0832	9.6727	955	912025	870983875	30.9031	9.8477
906	820836	743677416	30.0998	9.6763	956	913936	873722816	30.9192	9.8511
907	822649	746142643	30.1164	9.6799	957	915849	876467493	30.9354	9.8546
908	824464	748613312	30.1330	9.6834	958	917764	879217912	30.9516	9.8580
909	826281	751089429	30.1496	9.6870	959	919681	881974079	30.9677	9.8614
910	828100	753571000	30.1662	9.6905	960	921600	884736000	30.9839	9.8648
911	829921	756058031	30.1828	9.6941	961	923521	887503681	31.0000	9.8683
912	831744	758550528	30.1993	9.6976	962	925444	890277128	31.0161	9.8717
913	833569	761048497	30.2159	9.7012	963	927369	893056347	31.0322	9.8751
914	835396	763551944	30.2324	9.7047	964	929296	895841344	31.0483	9.8785
915	837225	766060875	30.2490	9.7082	965	931225	898632125	31.0644	9.8819
916	839056	768575296	30.2655	9.7118	966	933156	901428696	31.0805	9.8854
917	840889	771095213	30.2820	9.7153	967	935089	904231063	31.0966	9.8888
918	842724	773620632	30.2985	9.7188	968	937024	907039232	31.1127	9.8922
919	844561	776151559	30.3150	9.7224	969	938961	909853209	31.1288	9.8956
920	846400	778688000	30.3315	9.7259	970	940900	912673000	31.1448	9.8990
921	848241	781229961	30.3480	9.7294	971	942841	915498611	31.1609	9.9024
922	850084	783777448	30.3645	9.7329	972	944784	918330048	31.1769	9.9058
923	851929	786330467	30.3809	9.7364	973	946729	921167317	31.1929	9.9092
924	853776	788889024	30.3974	9.7400	974	948676	924010424	31.2090	9.9126
925	855625	791453125	30.4138	9.7435	975	950625	926859375	31.2250	9.9160
926	857476	794022776	30.4302	9.7470	976	952576	929714176	31.2410	9.9194
927	859329	796597983	30.4467	9.7505	977	954529	932574833	31.2570	9.9227
928	861184	799178752	30.4631	9.7540	978	956484	935441352	31.2730	9.9261
929	863041	801765089	30.4795	9.7575	979	958441	938313739	31.2890	9.9295
930	864900	804357000	30.4959	9.7610	980	960400	941192000	31.3050	9.9329
931	866761	806954491	30.5123	9.7645	981	962361	944076141	31.3209	9.9363
932	868624	809557568	30.5287	9.7680	982	964324	946966168	31.3369	9.9396
933	870489	812166237	30.5450	9.7715	983	966289	949862087	31.3528	9.9430
934	872356	814780504	30.5614	9.7750	984	968256	952763904	31.3688	9.9464
935	874225	817400375	30.5778	9.7785	985	970225	955671625	31.3847	9.9497
936	876096	820025856	30.5941	9.7819	986	972196	958585256	31.4006	9.9531
937	877969	822656953	30.6105	9.7854	987	974169	961504803	31.4166	9.9565
938	879844	825293672	30.6268	9.7889	988	976144	964430272	31.4325	9.9598
939	881721	827936019	30.6431	9.7924	989	978121	967361669	31.4484	9.9632
940	883600	830584000	30.6594	9.7959	990	980100	970299000	31.4643	9.9666
941	885481	833237621	30.6757	9.7993	991	982081	973242271	31.4802	9.9699
942	887364	835896888	30.6920	9.8028	992	984064	976191488	31.4960	9.9733
943	889249	838561807	30.7083	9.8063	993	986049	979146657	31.5119	9.9766
944	891136	841232384	30.7246	9.8097	994	988036	982107784	31.5278	9.9800
945	893025	843908625	30.7409	9.8132	995	990025	985074875	31.5436	9.9833
946	894916	846590536	30.7571	9.8167	996	992016	988047936	31.5595	9.9866
947	896809	849278123	30.7734	9.8201	997	994009	991026973	31.5753	9.9900
948	898704	851971392	30.7896	9.8236	998	996004	994011992	31.5911	9.9933
949	900601	854670349	30.8058	9.8270	999	998001	997002999	31.6070	9.9967